DEPARTMENT OF TRANSPORTATION

# Balanced Design of Asphalt Mixtures


**David Newcomb, Principal Investigator**
Texas A&M Transportation Institute


**June 2018**


Research Project
Final Report 2018-22

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A balanced mix design (BMD) sets a maximum asphalt content based on the rutting criterion and a minimum asphalt content based on cracking criterion. This project developed a BMD framework for the Minnesota Department of Transportation (MnDOT) and used it to evaluate materials from four Minnesota projects using the Illinois Flexibility Index, the IDEAL-CTAL-CT-CT Cracking Test Index by the Texas A&M Transportation Institute, and the Disk-shaped Compact Tension test for cracking and the Hamburg Wheel Tracking test for rutting. For the four test mixtures, the performance tests and the BMD procedure were successful in distinguishing the influence of asphalt content on cracking resistance and rutting resistance. There was fairly good agreement among the cracking tests for the asphalt content and only a slight deviation from volumetric asphalt content in most cases. The cracking and rutting performance criteria need to be refined for different applications based on characteristics such as climate, lift thickness, traffic level, and placement within the pavement structure.

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# BALANCED DESIGN OF ASPHALT MIXTURES

## FINAL REPORT

*Prepared by:*

David Newcomb, P.E., Ph.D.
Senior Research Engineer/Division Head
Texas A&M Transportation Institute

Fujie Zhou, P.E., Ph.D.
Research Engineer
Texas A&M Transportation Institute

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| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| APA | Asphalt Pavement Analyzer |
| BMD | balanced mix design |
| Caltrans | California Department of Transportation |
| CMOD | crack-mouth opening displacement |
| COV | coefficient of variation |
| DCT | Disc-Shaped Compact Tension |
| DOT | department of transportation |
| ESAL | equivalent single axle loads |
| FD | force-displacement |
| FHWA | Federal Highway Administration |
| FHWA-ALF | FHWA-accelerated load facility |
| FI | Flexibility Index |
| FN | flow number |
| HMA | hot mix asphalt |
| HWTT | Hamburg Wheel Track Test |
| IDEAL-CT | Indirect Tension Asphalt Cracking Test |
| IDT | Indirect tension |
| IDOT | Illinois Department of Transportation |
| I-FIT | Illinois Flexibility Index Test |
| LADOTD | Louisiana Department of Transportation and Development |
| LLD | load-load line displacement |
| LTOA | long-term oven aged |
| MnDOT | Minnesota Department of Transportation |
| NCHRP | National Cooperative Highway Research Program |
| NJDOT | New Jersey Department of Transportation |
| NMAS | nominal maximum aggregate size |
| OT | Overlay Tester |
| PG | Performance Grade |
| QC/QA | quality control/quality assurance |
| RAP | reclaimed asphalt pavement |
| RAS | reclaimed asphalt shingles |
| RBR | recycled binder replacement |
| SCB | Semi-Circular Bend |
| SGC | Superpave Gyratory Compactor |
| SIP | stripping inflection point |
| SN | Stripping number |
| STOA | short-term oven aged |
| TTI | Texas A&M Transportation Institute |
| TxDOT | Texas Department of Transportation |
| WisDOT | Wisconsin Department of Transportation |
| WMA | warm mix asphalt |

# EXECUTIVE SUMMARY

The ideal asphalt mixture has been studied, tested, and tried since the 1860s. As a result, asphalt mixture performance tests have also evolved. Since the 1990s, asphalt research has focused on binders, aggregate gradations and shapes, laboratory compactive effort, and the field performance of mixtures. The use of performance testing was severely diminished during this period in favor of volumetric testing to define the optimum proportioning of asphalt mixtures. When oil prices surged at the end of the 2000s, agencies and contractors began using more recycled asphalt pavement and recycled asphalt shingles in their mixes. Long-used additives such as polyphosphoric acid and re-refined engine oil bottoms were used in greater quantities. But these new trends also came with compatibility problems and ultimately performance issues in terms of durability, rutting, and cracking.

Researchers developed the balanced mix design (BMD) approach to address rutting and cracking. A BMD sets a maximum asphalt content according to a rutting criterion and the minimum asphalt content by the cracking criterion. This project developed a BMD framework for the Minnesota Department of Transportation (MnDOT) and used it to evaluate materials from Minnesota projects.

The proposed MnDOT BMD is:

1. Select the materials for use according to the current practice. Aggregates should meet the consensus properties and gradation required for the particular application, and the asphalt grade should be selected according to MnDOT PG binder guidelines.

2. Combine materials, mix, and short-term oven age (STOA) for 2 hours for the rutting test and long-term oven age (LTOA) for 4 hours for the cracking test at the suggested compaction temperature.

3. Using a volumetric design, define the asphalt content ($AC_v$) meeting the requirement of 4.0 percent air voids at $N_{design}$.

4. Prepare samples at $AC_v$, $AC_v$ +0.5 percent, and $AC_v$ −0.5 percent.

5. After aging, compact samples to 7±0.5 percent air voids. This level of target air voids represents what might be expected in field compaction.

6. Conduct cracking and rutting performance tests.

7. Select the asphalt content defined as the balanced asphalt content ($AC_B$) according to the test results and accounting for the allowable variance of asphalt content in construction. Adding construction tolerance ensures that the resulting field mixture does not fall below the minimum required by the cracking performance testing.

For the four test mixtures, the performance tests and the BMD procedure were successful in distinguishing the influence of asphalt content on cracking resistance and rutting resistance. There was fairly good agreement among the cracking tests for the asphalt content for non-carbonate aggregates and only a slight deviation from volumetric asphalt content in most cases. The carbonate mixes seemed to be better suited for lower-volume roads, and the non-carbonate mixes seemed better suited for higher-volume roads. The cracking and rutting performance criteria need to be refined for different applications based on characteristics such as climate, lift thickness, traffic level, and placement within the pavement structure.

Future research needs include:

- Failure criteria for all the cracking tests
- Allowable tolerance for asphalt content during production
- A laboratory standard for aging mixtures in mix design to ensure an adequate level of aging
- Cracking criteria validated on a large number of roads with different traffic, climate, and soil conditions
- A process for introducing cracking tests into Quality Control/Quality Assurance (QC/QA)
- Eliminating as much variability as possible for the sake of precision and bias

The next steps in the implementing the BMD are to:

1. Develop criteria for the cases in which it is to be used

2. Determine if the pass/fail criteria need to be adjusted for differences in factors

3. Construct monitored field sections to determine the relationship between the use of BMD and field performance

4. Refine the procedure and performance criteria based on the results

The most important factors to consider in implementation are pavement structure, climate, traffic, mix design, and QC/QA testing.

# CHAPTER 1: INTRODUCTION

## 1.1 BACKGROUND

The earliest attempt to design asphalt mixtures dates back to the 1860s, according to Campbell (1989). At that time, bituminous pavements were being constructed on the East Coast, notably in Washington, D.C. The binder in these pavements was either tar or Trinidad Lake asphalt. These early mixtures were for sheet asphalt pavements that were comprised of simply sand and binder, and the performance of these pavements was mostly substandard due to rutting. This was largely due to the overabundance of fines in the mixtures and the inconsistent hand mixing that was used to produce them (Roberts et al. 1996). It was not until the early 1900s that Clifford Richardson began investigating the composition of mixtures and determined the importance of voids in mineral aggregate and air void content to the stability and durability of the mixtures (Richardson 1912). Richardson developed the first known mixture test method, the Pat Test, in which a sample of freshly mixed asphalt and aggregate was applied to brown manila paper. A dark stain indicated a mix with too much asphalt and a light stain indicated insufficient asphalt.

In the early 20th century, the Warren Brothers introduced a patented asphalt mix referred to as Warren Bitulithic. This was essentially a recipe mix design calling for prescribed proportions of fine aggregate, coarse aggregate, and asphalt. The mix was simply combined in a batch plant (Figure 1.1) and placed on the roadway. The mixtures did not always turn out as envisioned (Figure 1.2) (Mahoney 2002). Later on, tests were developed for binders and mixtures. One example of an early mixture performance test was the Hubbard-Field test, which used a punching shear loading to evaluate mix strength (Roberts et al. 1996). In 1938, Bruce Marshall developed the Marshall method of mix design. His approach combined volumetric measurements with circumferential compression test in which the peak load (stability) and the vertical displacement at peak load (flow) were measured and recorded (Leahy and McGinnis 1999). Also, in the 1930s, Francis Hveem, of the California Highway Department, devised a mixture design method that accounted for asphalt absorption and employed a triaxial test that measured strength and volumetric displacement (stability) (Leahy and McGinnis 1999). Both the Marshall stability test and the Hveem stability test were performed at a temperature of 140 ° F (60 ° C), which was thought to represent the maximum pavement temperature to which a mix might be exposed. Thus, they addressed the behavior of the mix for rutting. These latter two approaches were the mainstays of asphalt mix design procedures throughout most of the 20th century.


**Figure 1.1. Early Hot Mix Plant (circa 1912) (Mahoney 2002).**


**Figure 1.2. Example of Early Unstable Asphalt Mixture (circa 1912) (Mahoney 2002).**

In 1988, the Strategic Highway Research Program began with a strong emphasis on asphalt pavements with research funding of $50,000,000 (McDaniel et al. 2011). The Superpave system resulted in an improved binder specification that required binder testing at low service temperature, high service temperature, and at the construction temperature. New aging protocols were introduced to more realistically reflect the state of the material in service. This new approach to asphalt mixture design required the selection of quality materials, a new laboratory compaction method, and volumetric criteria. Material quality criteria and compaction levels were related to the expected traffic loading. More emphasis was placed on aggregate structure for rutting resistance. A simple shear tester, later referred to as the Superpave shear tester (Figure 1.3), was introduced to evaluate the permanent deformation behavior of asphalt mixtures.

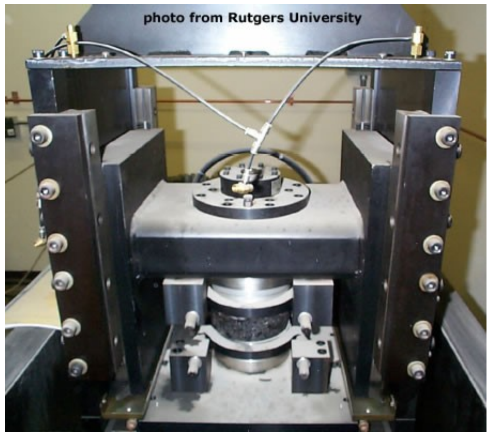



**Figure 1.3. Superpave Shear Tester (from PavementInteractive.org).**

A massive implementation program followed the development of Superpave in 1992 with many states and local agencies. Over the course of the next 12 years, research was conducted on many fronts including binders, aggregate gradations and shapes, laboratory compactive effort, and the field performance of mixtures. Many of the concepts originally assumed to be valid in the original development of Superpave were subsequently changed on the basis of these studies. For instance, the notion that a restricted zone in the aggregate gradation was needed to prevent rutting was shown as being unnecessary (Kandhal and Cooley 2003a). Also, the study showed that coarse aggregate gradations would perform better than fine gradations in resisting rutting was dispelled as it was shown that aggregate shape was a more important factor and that coarser mixes tended to crack easier than fine mixes (Kandhal and Cooley 2003b, Epps et al. 2002). Gyratory compaction efforts were reduced when it was discovered that the compaction was crushing aggregate and producing dry mixtures (Prowell and Brown 2007).

The Superpave shear tester was intended to be the performance test for the Superpave system but the implementation did not go beyond federally purchased devices located in the regional Superpave centers for the most part. The cost of the device in the 1990s was about $250,000 and thus beyond the financial means of most state departments of transportation (DOTs) laboratory budgets. Furthermore, the criteria suggested for permanent deformation at various levels of traffic were not widely validated. Thus, performance testing did not become a part of the final mix design procedure adopted by states. Instead, simpler rutting tests such as the Asphalt Pavement Analyzer (APA) and the Hamburg Wheel Track Test (HWTT) were adopted by some states, and most states had provisions for moisture sensitivity testing using American Association of State Highway and Transportation Officials (AASHTO) test method T 283. Many states simply relied on volumetric design approaches with no provision for cracking or rutting performance testing.

Initially, the lack of a performance test or solely relying on a rutting test within the Superpave system worked well most of the time. While the use of reclaimed asphalt pavement (RAP) was pervasive during the 1990s, the implementation of Superpave kept RAP content in mixtures at a minimum. This was because agencies wanted to understand the Superpave system without the confounding presence of high RAP quantities in mixtures. Also, while the use of polymer binders was beginning, their use was not as wide spread as it would become in the 2000s.

Toward the middle and end of the 2000s oil prices began significantly increasing, reaching a peak in 2008. As the price increased, the asphalt mixture industry sought relief by asking agencies to increase the amount of allowable RAP and reclaimed asphalt shingles (RAS) in their products. The recycled binder replacement (RBR) from RAP and RAS began reaching levels of 30 percent or more, which allowed the asphalt industry to remain competitive. At the same time, some oil companies producing asphalt developed means to produce more valuable refinery products from asphalt through an increased use of a process known as coking. Also, solvent deasphalting technology was used to a greater extent, which allowed a post-production method of removing more valuable petroleum fractions from the asphalt. Coking and solvent deasphalting caused asphalt prices to increase at a rate even greater than petroleum itself. Deasphalted petroleum bottoms began to be sold to blending plants, which combined the bottoms with lighter petroleum fractions to produce asphalt that could be sold for road construction. While this did not create issues on a widespread basis, if the blending plant used the wrong type of diluent the chemical balance of the asphalt could be upset creating compatibility problems and ultimately performance issues in terms of durability, rutting, and cracking. With these problems becoming pervasive in recent years, state DOTs have sought to address them with various performance tests for rutting and, more recently, cracking. The use of a rutting test to define a maximum asphalt content beyond which would likely result in permanent deformation failures and a cracking test that would indicate a minimum asphalt content below which could result in cracking failure from the framework for what is known as a balanced mix design (BMD).

## 1.2 OBJECTIVE

This research project developed a framework for BMD for the Minnesota Department of Transportation (MnDOT) and used the principles of BMD for evaluating materials from actual Minnesota projects.

## 1.3 SCOPE

This project consisted of a literature review and current technology review to synthesize the states of practice for asphalt performance testing and BMD. From the information gained and discussions with the MnDOT Technical Advisory Panel, the research team formulated a plan for BMD. Materials from four different mixes in Minnesota were sampled by MnDOT and shipped to the Texas A&M Transportation Institute (TTI) for evaluation. The materials were combined into mixtures and tested in three different cracking tests and one rutting test. The results of the tests were analyzed, and an optimum asphalt content was determined both volumetrically and by the proposed BMD approach. Along with an analysis of the results, the variability of the cracking test methods were determined and presented as well.

# CHAPTER 2: LITERATURE REVIEW

## 2.1 BALANCED MIX DESIGN

The changing nature of the asphalt binder with increased RBR, increased use of polymers, and blending modifications created the necessity for agencies to look beyond volumetric asphalt mixture design and incorporate performance testing for rutting and cracking. As performance testing became more extensively used, the development of BMDs came into being. One of the first of these was developed in Texas in the mid-2000s by Zhou and his co-workers (2006). In this approach, the need to address cracking and rutting performance was solved by setting a maximum asphalt content where the rutting criterion was exceeded and setting the minimum asphalt content by the failure point of the cracking criterion as shown in Figure 2.1. BMD improves the probability that an asphalt mixture will have the combination and quality of ingredients to resist deterioration from rutting, cracking, and moisture damage. The goal of the BMD is to achieve the combination and proportions of binder, aggregate, and other ingredients to pass the criteria of performance tests for permanent deformation and cracking types for a given level of traffic, climate, and pavement structure.

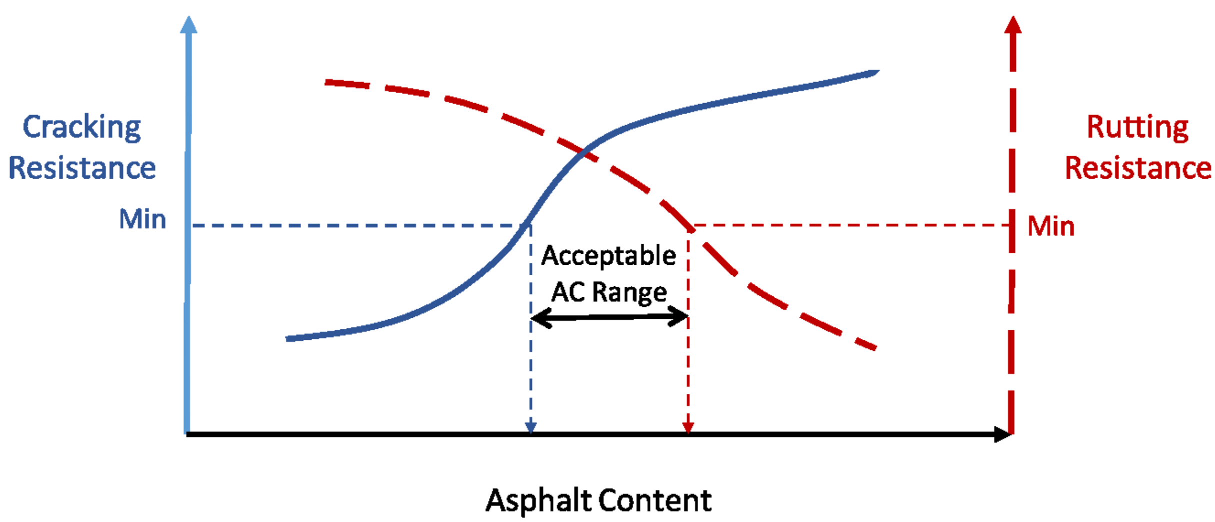



**Figure 2.1. Concept of BMD.**

Recently, National Cooperative Highway Research Program (NCHRP) Synthesis 492 "Performance Specifications for Asphalt Mixtures" (McCarthy et al. 2016) showed that many DOTs share a belief that the current asphalt mix design procedures do not ensure performance. Mohammad et al. (2016) conducted a separate survey on the "State of Balanced Mixture Design Practice" and reported that 21 out of 27 DOTs include laboratory mechanical tests in their mixture design specifications. The most common test was for moisture damage. A majority of those states (14 out of 21) are using a rut test (either the APA or HWTT) to indicate rutting potential. Most do not yet have a test for cracking resistance.

The Federal Highway Administration (FHWA) Expert Task Group on Asphalt Mixture and Construction formed a Task Force on BMD in 2015 to discuss possible changes to asphalt mixture design to incorporate performance tests and encourage the implementation of BMD. According to this group, BMD is defined as "Asphalt mixture design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mixture aging, traffic, climate and location within the pavement structure" (FHWA 2016). This group developed and submitted a Research Needs Statement that was considered by the AASHTO Subcommittee of Materials.

Tim Aschenbrener of FHWA identified three different approaches (Aschenbrener 2016) in the way that DOTs and FHWA are using or are considering using BMD. The basic difference in these schemes is the interaction between volumetric considerations and performance testing in determining the target asphalt content. The three approaches are illustrated in Figure 2.2 (Hall 2016), and they consist of:

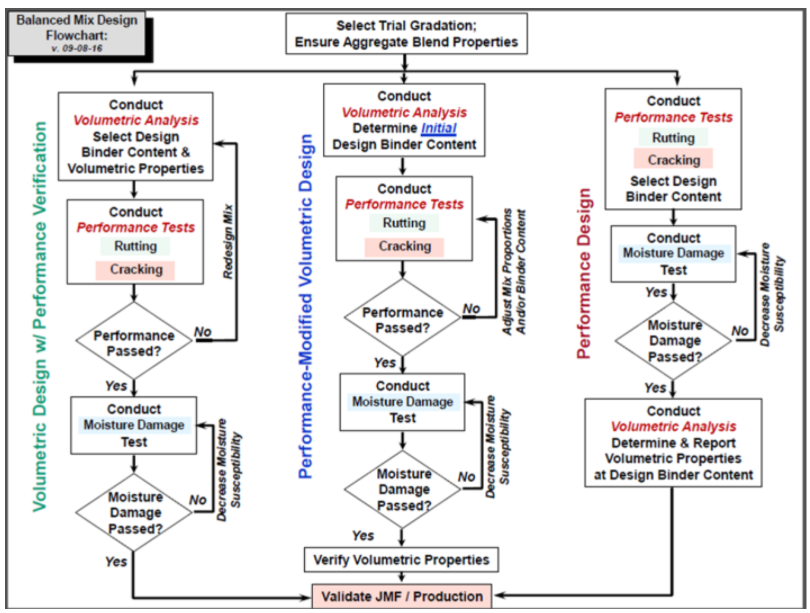



Figure 2.2. Options for BMD Approaches (Hall 2016).

1. **Volumetric design with performance verification**. This is the most commonly used method employed by DOTs. A volumetric design is accomplished first followed by performance testing of the mixture at the target asphalt content. The volumetric and performance testing criteria must both be satisfied before the design is considered complete. If improvements in the mixture must be made, then adjustments to aggregate source, aggregate gradation, asphalt source, or other additives must be made. Once the mixture has passed the rutting and cracking criteria and satisfied the volumetric requirements, it is then subjected to a moisture sensitivity evaluation that must be passed before becoming the job mix formula.
2. **Performance-modified volumetric design.** As shown in Figure 2.2, this approach begins with a volumetric evaluation of the asphalt-aggregate combination to determine a starting point for the asphalt content. Performance testing for rutting and cracking are then conducted, and if the mixture fails either of these two criteria, the asphalt content or mixture proportions are

adjusted to meet the performance criteria exclusive of the volumetric requirements. A moisture sensitivity test is performed to ensure the durability of the mix. Thus, the volumetric design is only used to provide a starting point.

3. **Performance design.** In a performance design, there is limited or no initial consideration of the volumetric requirements. Minimum criteria might be set for binder properties or aggregate properties but the objective would be to use the components in proportions that would meet the performance test criteria. As shown in Figure 2.2, performance testing would be used to establish the mix component proportions. Volumetric properties such and air voids, voids in mineral aggregate, and minimum asphalt content and aggregate gradation might be considered as recommendations rather than mandatory parameters. This type of a BMD has not been used by DOTs as of this report, but could become a standard procedure at some point in time.

Mixture aging conditioning will be an important feature of BMD. Currently, aging protocols are being studied for both short-term and long-term simulation. Often, short-term aging is used for rutting tests to preclude failures due to permanent deformation early in the pavement life. NCHRP Project 9-52 (Newcomb et al. 2014) found that two-hour conditioning of asphalt samples at 240 ° F (116 ° C) for warm mix asphalt (WMA) and 275 ° F (135 ° C) for hot mix asphalt (HMA) provided an adequate representation of mixture aging prior to placement and compaction. However, the long-term aging protocol of two hours at 275 ° F (135 ° C) plus five days at 185 ° F (85 ° C) was only successful in mimicking about one year of aging in warm climates and about two years in cold climates. NCHRP Project 9-54 is currently defining long-term aging protocols for laboratory mixes. It is recommended that crack testing of mixtures adhere to some form of long-term aging protocol for mixture design since cracking becomes more critical as the mixture ages.

Performance testing of asphalt mixtures is the core issue in a BMD. As presented in Figure 2.1, the normal battery of tests includes rutting resistance, cracking resistance, and moisture sensitivity resistance. The concept of balance comes from the notion that an asphalt content that will just meet the rutting criterion will establish a maximum and that an asphalt content that will just meet a cracking criterion will represent a minimum. An asphalt content that lies between the minimum and maximum will represent a balance between the two.

## 2.2 PERFORMANCE TESTING

Performance testing is central to the development of a BMD protocol, and there are a number of considerations in the selection of the tests to be used in mixture design (Zhou et al. 2015). As determined in a workshop of agency, industry, and research personnel, the primary issues for cracking tests are the relationships between test results and performance closely followed by the sensitivity of the test to mixture parameters. Beyond these two fairly obvious needs, test simplicity for mix design and quality control/quality assurance (QC/QA) and low variability were rated highly as well. Rutting tests are well established with the HWTT, APA, and, to a lesser extent, the Asphalt Mixture Performance Tester parameter of flow number (FN) being used to characterize permanent deformation behavior.

## 2.3 CRACKING TESTS

Although there are a number of tests that exist to measure cracking resistance, the authors have chosen those that are most likely to represent thermal cracking behavior and that can be reasonably expected to be successfully incorporated into asphalt mixture design and possibly assurance testing. The tests that have been selected for inclusion in this review are the Disc-Shaped Compact Tension (DCT) test, the Semi-Circular Bend (SCB) tests (University of Minnesota and Illinois methods), the Texas Overlay Tester (OT), and the Indirect Tension test.

### 2.3.1 Disc-Shaped Compact Tension Test

Buttlar and his co-workers (Wagoner et al. 2006) developed the DCT test for characterizing cracking resistance of asphalt mixtures at low temperatures. Currently, the DCT test is found in ASTM standard test method D7313, and the MnDOT has developed an alternative version. A disk-shaped specimen (Figure 2.3) is pulled apart until the post peak level has reduced to 0.02 lb (0.1 kN). The geometry of the specimen is a 6-in. (150-mm) diameter, 2-in. (50-mm) thick overall dimension with two 1-in. (25 mm) holes on either side of a 2.46-in. (62.5 mm) notch cut into a flattened portion of the circumference. The DCT is often conducted at 18 $^{o}$ F (10 ° C) warmer than the PG low temperature grade in a crack-mouth opening displacement (CMOD) controlled mode with an opening rate of 0.04 in/min (1 mm/min). MnDOT performs DCT testing at site-specific temperatures, according to the 98% reliability LTPPBind low temperature, plus 10$^{o}$C. Figure 2.3 also shows a typical test curve. The fracture energy (*Gf*) is calculated by determining the area under the Load-CMOD curve normalized by the initial ligament length and thickness. The larger the *Gf*, the better the cracking resistance of the asphalt mixture is. The typical coefficient of variation (COV) for the DCT test for virgin mixtures is around 10 percent, which is fairly low. When dealing with RAP/RAS mixtures, the COV may be expected to exceed 15 percent.

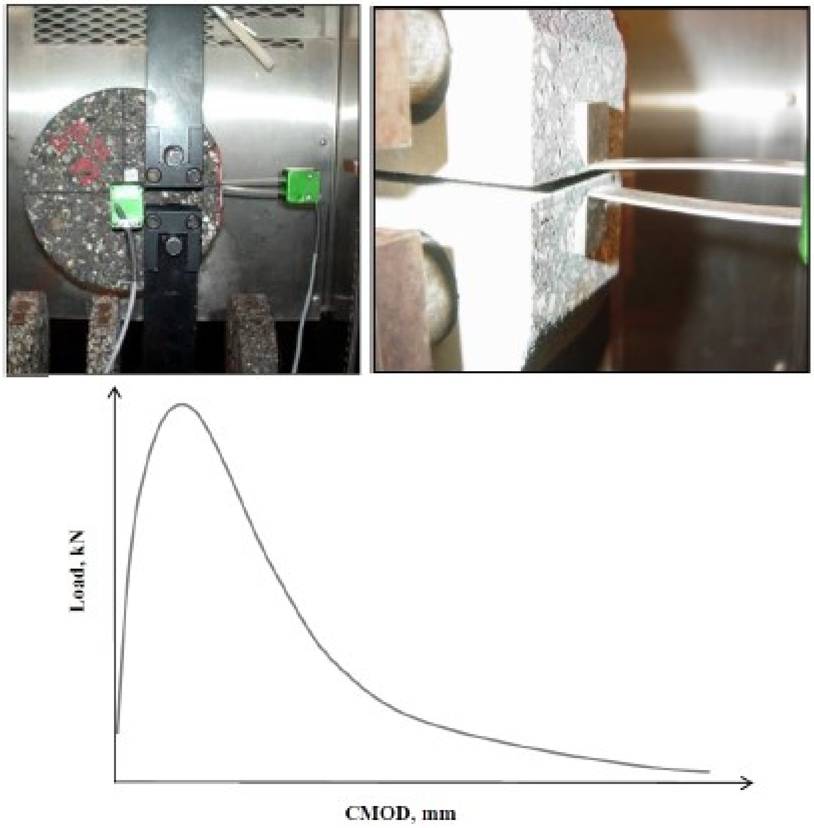
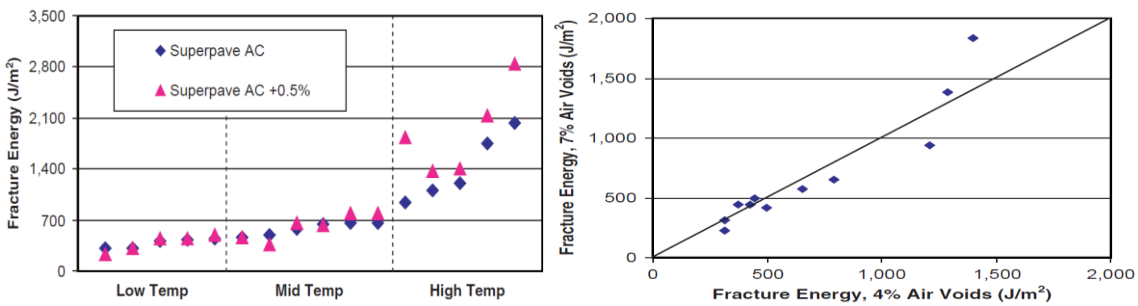


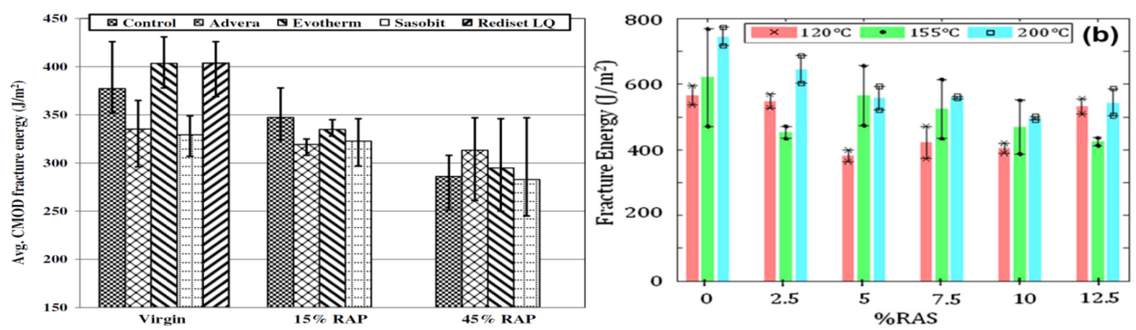

**Figure 2.3. DCT Test Setup, CMOD Gauge, and Typical Test Curve (Marasteanu et al. 2012).**

The major concern for the DCT test is the specimen preparation, although the specimen can be easily made using the Superpave Gyratory Compactor (SGC) or field cores. The DCT sample preparation involves four cuts (two cuts to make 2-in. (50-mm) thick sample, one cut to create a flat surface for CMOD gauge, and one cut for the 2.46-in. (62.5 mm) notch) and two coring operations for the tension holes. Researchers at the University of Illinois have determined the average fabrication time per specimen to be in the 10- to 15-minute range for DCT testing, which includes the four saw cuts and two cored holes. This is based upon production of at least a dozen test specimens. The fabrication of fewer test specimens will obviously lead to a shorter per-specimen preparation time. The CMOD gauge needs to be mounted to the two sides of the crack mouth, which is easy and fast. The testing time for DCT is short. Although the DCT test itself takes only 1 to 6 minutes to perform, the actual amount of test time per specimen is probably more like 15 minutes, accounting for stabilization of test temperature and loading samples into the test apparatus (Marasteanu et al. 2012). Performing the DCT test requires little technician training if the commercially available DCT tester with integrated operating software is employed. Currently, Testquip LLC manufactures the DCT test equipment specifically for running ASTM D7313-13. Alternatively, a universal servo-hydraulic testing system equipped with an environmental chamber can be used to perform the DCT test.

Calculating the DCT fracture energy ($G_f$) is relatively easy, but a data analysis program or Excel Macro is needed, since integration of the curve is involved. It is very easy to interpret the DCT test results ($G_f$), which involves comparing the results with the established pass/fail criterion for thermal cracking. Braham et al. in Marasteanu et al. (2007) reported results for 28 asphalt mixtures designed for cold climates and investigated four parameters: 1) aggregate type (limestone and granite); 2) test temperature -3.6 °F( −2 ° C) below the low temperature grade [low temperature], 18 ° F (10 ° C) above the low temperature grade [mid-temperature], and 3.6 ° F (2 ° C) above the low temperature grade [high temperature]); 3) asphalt content (design asphalt content and design asphalt content plus 0.5 percent); and 4) air voids (4 percent and 7 percent). The DCT fracture energy is sensitive to binder content at higher temperatures, aggregate type, and temperature, but not sensitive to asphalt content at low and mid-temperatures and the air voids, as shown in Figure 2.4. This finding was later confirmed by Dave et al. (2011). Dave et al. (2011) also found that aging had limited effect on fracture energy when aging is induced using the AASHTO R30 protocol. Recently, Hill et al. (2013) found that the inclusion of RAP led to reduced DCT fracture energy and consequently potentially increased thermal cracking irrespective of the WMA additive employed. Arnold et al. (2014) concluded that the mixtures containing RAS had lower DCT fracture energies. Thus, the DCT test is sensitive to the presence of recycled materials (RAP and RAS).




(a) Fracture energy vs. test temperature, asphalt content, and air voids (Braham et al. 2007)




(b) Fracture energy vs. RAP (Hill et al. 2013) (c) Fracture energy vs. RAS (Arnold et al. 2014)

**Figure 2.4. Sensitivity of DCT Test to Asphalt Mix Composition and Design Parameters.**

Under the national pooled fund study, *Investigation of Low Temperature Cracking in Asphalt Pavements – Phase II*, field thermal cracking data were correlated to DCT fracture energy (Marasteanu et al. 2012). Figure 2.5 shows such correlation. From these results (Figure 2.5), a minimum of fracture energy of 400 J/m$^2$ is suggested for protection against thermal cracking. Fracture energy in the range of 350–400 J/m$^2$ is considered borderline, and may be permissible on less critical projects, where a low to moderate degree of thermal cracking can be tolerated. For critical projects, a factor of safety can be achieved by specifying a minimum fracture energy of 600 J/m$^2$.

A thermal cracking specification was proposed by Buttlar et al. for asphalt mix design (Marasteanu et al. 2012). Since the DCT test results presented in Figure 2.5 were from on cores taken out of older pavements, a 15 percent increase in fracture energy was proposed in the pooled fund study to take into account the fact that these requirements are specified for laboratory-mixed, laboratory-compacted mixtures with short-term aging. Table 2.1 provides specification limits for three levels of project criticality. Note that the specification applies for surface mixes only.

Additionally, the DCT fracture energy ($G_f$) combining with other viscoelastic properties of asphalt mixtures (such as creep compliance) also can be used as inputs to a mechanistic model (such as ILLI-TC) to predict thermal cracking development of asphalt pavements.

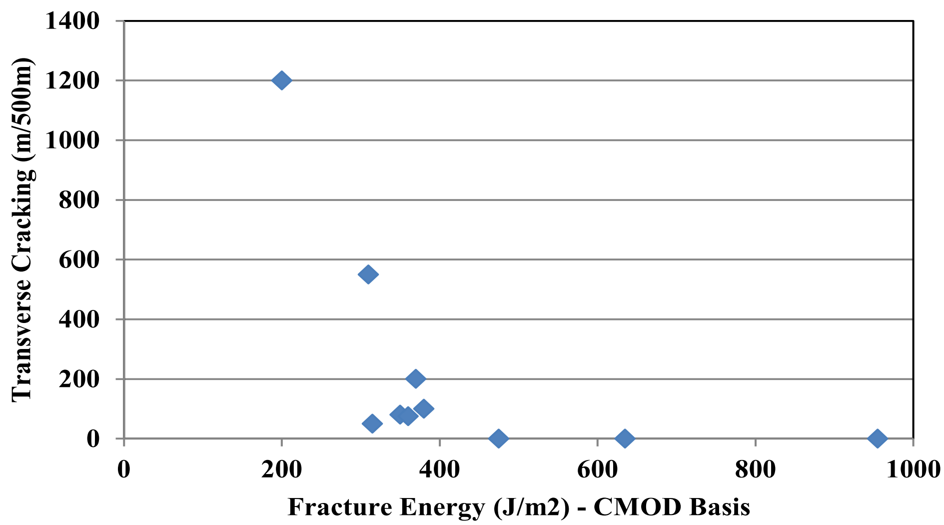



**Figure 2.5. Correlation between DCT and Thermal Cracking (Marasteanu et al. 2012).**

**Table 2.1. Recommended Low Temperature Cracking Specification for Loose Mixes (Marasteanu et al. 2012).**

| Contents | Project Criticality/Traffic Level | | |
|---|---|---|---|
| | Low<br><10M ESALs | Moderate<br>10–30M ESALs | High<br>>30M ESALs |
| Minimum Fracture Energy (J/m$^2$)@low-temperature PG+10 ° C | 400 | 460 | 690 |

ESAL = equivalent single axle loads.

### 2.3.2 Semicircular Bend Test (Minnesota)

The SCB test for low temperature cracking was developed by Marasteanu and his co-workers (Li and Marasteanu 2004, Marasteanu et al. 2012). Currently, the SCB test is an AASHTO provisional standard test: AASHTO TP105-13. Similar to the DCT test, this version the SCB test characterizes the fracture energy of an asphalt mixture specimen. The test is conducted at 10 ° C warmer than the PG low temperature grade. Also similar to the DCT test, the SCB test is run in a CMOD controlled mode. Figure 2.6 shows the SCB test setup and typical test results.

The SCB uses a 1-in. thick specimen with a CMOD rate of 0.03 mm/min, which is 33 times slower than the DCT loading rate. This increases the duration of the test to as much as 30 minutes. The SCB fracture energy ($G_f$) is calculated by determining the area under the load-load line displacement (LLD) curve normalized by the initial ligament length and thickness. Note that LLD is measured using a vertically mounted Epsilon extensometer. The CMOD measurement is used for maintaining the test stability in the post peak region of the test rather than calculating fracture energy.

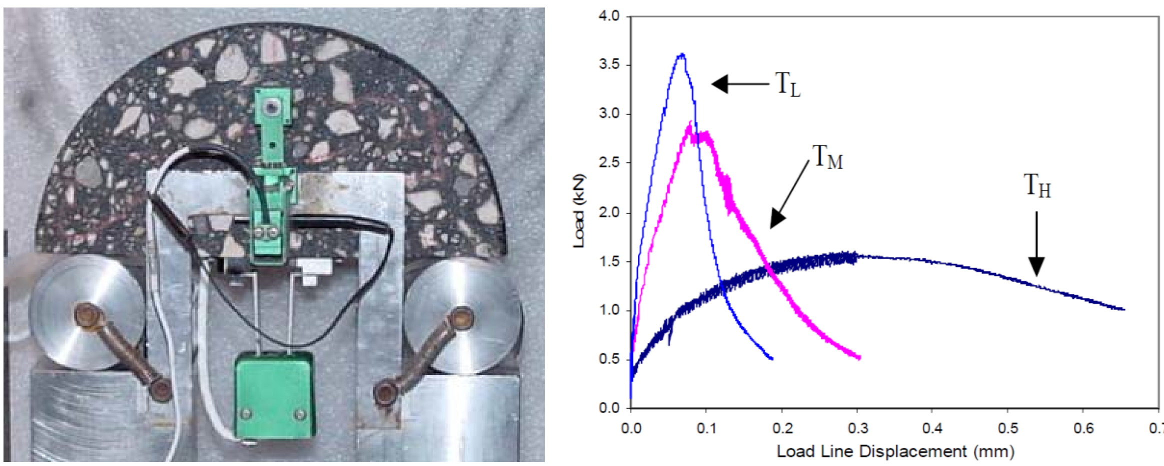



**Figure 2.6. SCB Test Setup, LLD Extensometer, and Typical Plot of SCB Test at Different Temperatures: TL-12 ° C below TM, TM-binder PG low limit+10 ° C, and TH-12 ° C above TM (Li and Marasteanu, 2004).**

The typical COV associated with this version of SCB testing is around 20 percent (Marasteanu et al. 2012).

The SCB specimen can be made from laboratory compacted specimens or field cores (Figure 2.7). Basically, it requires four cuts and two notches to obtain two SCB specimens, but no holes are required in the specimen.

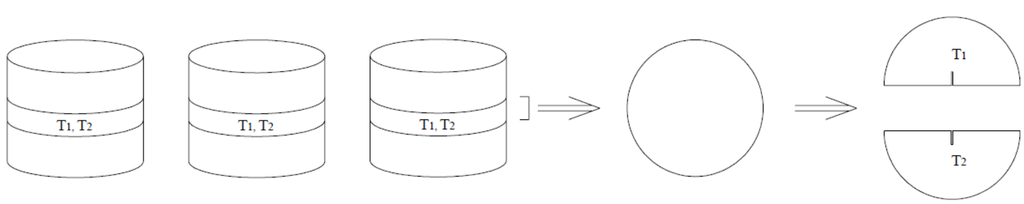



**Figure 2.7. SCB Specimen Preparation (AASHTO T105-13).**

The installation of CMOD gauge is the same (or similar) as the DCT test. Additionally, an Epsilon extensometer is mounted on the SCB specimen to measure the LLD for calculating fracture energy.

Running the SCB test includes four steps: 1) contact loading, 2) seating load, 3) three small amplitude loading cycles, and 4) fracture test step. It is not difficult to run the SCB test with commercially available test equipment with the designated software that integrates all these test steps. Similar to the DCT test, the SCB fracture energy can be directly compared with the established pass/fail criterion for thermal cracking.

Li and Marasteanu (2004) investigated the influence of asphalt binders (PG58-40, PG58-34, and PG58-28) used in MnROAD on SCB fracture energy, and found that the SCB fracture energy is sensitive to asphalt binder grade. At -22 ° F (−30 ° C), the mixture with PG58-40 binder had the highest fracture energy and the one with PG58-28 had the lowest. Li et al. (2008) later studied the impact of aggregate type (granite vs. limestone), air voids (4 percent vs. 7 percent), and asphalt content (optimum asphalt content vs. optimum asphalt content + 0.5 percent) on fracture energy. They found that the SCB fracture energy is sensitive to the aggregate type and the air voids, but not to asphalt content. The mixtures with granite aggregate had higher fracture energy than those with limestone, all other factors being constant; higher air voids were likely to result in mixtures with lower fracture energy. However, richer mixtures (higher asphalt content) do not necessarily result in higher fracture energy. Additionally, the impact of RAP contents on SCB fracture energy was evaluated by Li et al. (2008) and West et al. (2013). Li et al. (2008) found that the control mixtures (0 percent RAP) had the higher fracture energy and while 20 percent RAP mixtures exhibited similar fracture resistance to the control mixtures; the 40 percent RAP mixtures had significantly lower fracture resistance at the low temperature (Figure 2.8). However, recent results from NCHRP Project 9-46 (West et al. 2013) show that the SCB fracture energy does not provide a consistent result for mixtures with high RAP contents. They found that the SCB fracture energy was not significantly affected by RAP content except in one particular case.

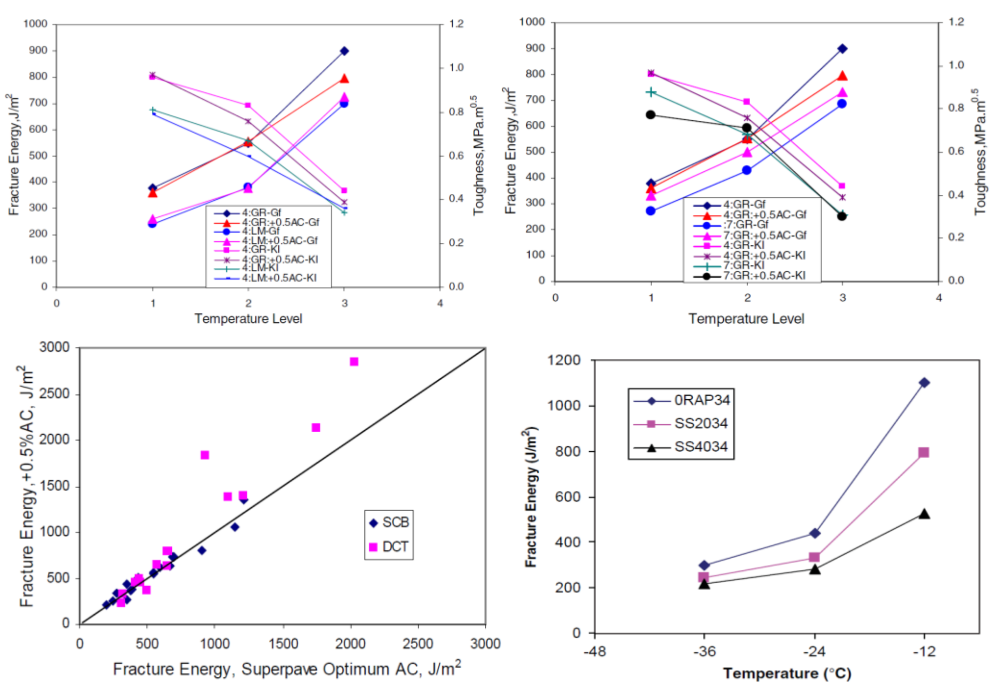



**Figure 2.8. Sensitivity SCB Fracture Energy to Mix Compositions and Design Parameters (Li et al. 2008).**

The same DCT test equipment manufactured by Testquip LLC can be used to run the SCB test with the addition of the SCB fixture. Alternatively, a universal servo-hydraulic testing system equipped with an environmental chamber can be used to perform an SCB test.

Similar to the DCT, the SCB fracture energy is correlated with the total length of transverse cracking observed in the field test sections in Illinois, Minnesota, and Wisconsin (Marasteanu et al. 2012) as shown in Figure 2.9. Based on the results plotted in Figure 2.9, a limiting value of 350 J/m$^2$ was proposed. This value may be adjusted to a limit of 400 J/m$^2$ to account for aging effects.

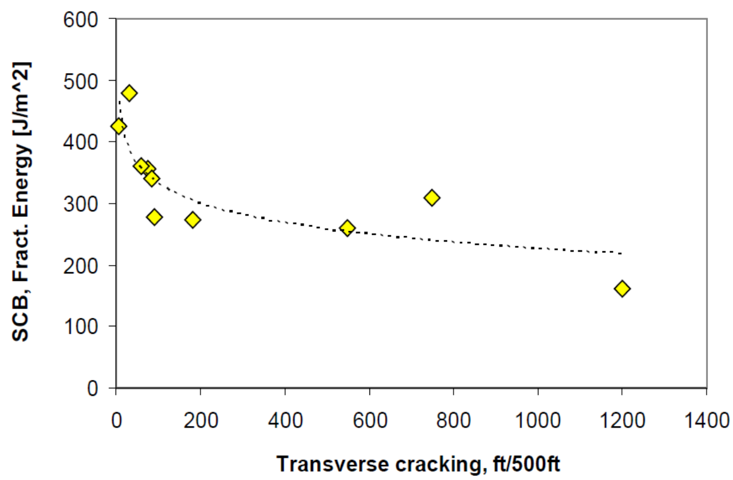



**Figure 2.9. Field Data Suggesting a Minimum SCB Fracture Energy of 350 J/m$^2$ to Prevent Thermal Cracking (Marasteanu et al. 2012).**

### 2.3.3 Illinois Flexibility Index Test (I-FIT)

A variation of the SCB test for thermal cracking has been proposed by the University of Illinois (Al-Qadi et al. 2015). It differs from the above version of SCB testing in that the vertical displacement of the loading head is used in place of the LLD and that the test is performed at 25 ° C and at a vertical crosshead speed of 50 mm/min. Because fracture energy can fail to differentiate between a strong, brittle mixture and a weak, ductile mixture, a new parameter called the Flexibility Index (FI) was introduced. The equation for the FI is:

$$FI = A\left(\frac{G_f}{abs(m)}\right)$$

Where: FI = Flexibility Index.

A = Calibration coefficient (0.01 for unaged mixtures).

$G_f$ = Work of the fracture energy ($W_f$) to peak load.

Abs(m) = Absolute value of the post-peak slope of the inflection point.

Figure 2.10 presents these parameters graphically. The higher the FI, the greater the cracking resistance is. Thus, a high FI value may be obtained through a combination of high fracture energy and low slope.

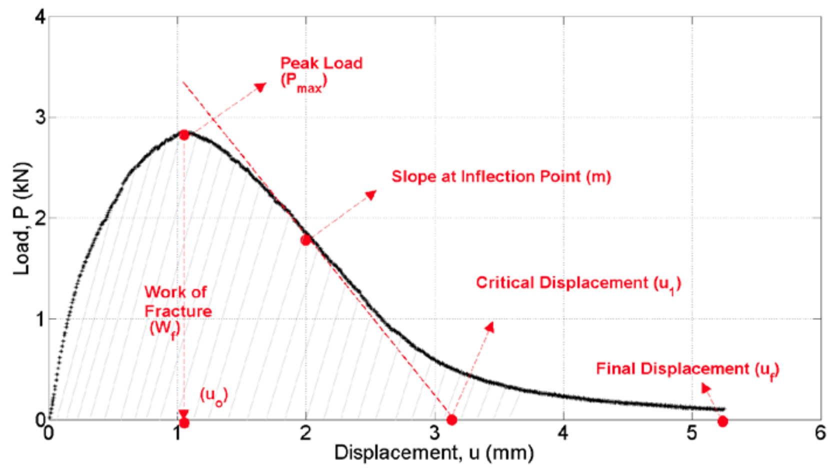



**Figure 2.10. Parameters for Determining the Flexibility Index.**

Mixture parameters leading to a higher FI value include the use of polymer modified asphalts and lower RBRs. COV in test results ranged from 4.2 percent to 21.3 percent with an average of about 10 percent (Al-Qadi et al. 2015). In a 2016 field study, Lippert et al. (2016) found that there was a correlation between lower transverse cracking and higher values of FI. The Illinois Flexibility Index Test (I-FIT) procedure is found in AASHTO TP124 "Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend (SCB) Geometry at Intermediate Temperature."

### 2.3.4 Texas Overlay Tester

Zhou and Scullion (2005) modified the OT, which had been widely used to evaluate the effectiveness of different geosynthetic materials since it was originally designed by Lytton et al. in the late 1970s (Germann and Lytton 1979) and proposed its use in evaluating cracking resistance of HMA overlays (Zhou and Scullion 2005; Zhou et al. 2006). Since then, different researchers including Bennert (2009), Bennert et al. (2009), Hajj et al. (2010), Bennert et al. (2011), and Walubita et al. (2012) have used the OT and have rated it as a reliable and practical test for screening and evaluating the crack resistance of HMA in the laboratory. Loria-Salazar (2008) did a comprehensive literature review study that lists different potential laboratory tests that have been in practice to evaluate the resistance of HMA to reflective cracking. He concluded that the OT is the only laboratory test method to undergo field validation that exhibited consistency between the laboratory test results and their corresponding field performance.

The OT was designed by Germann and Lytton (1979) to simulate the opening and closing of joints or cracks, which are the main driving force inducing reflective crack initiation and propagation. The key parts of the apparatus consist of two steel plates, one fixed and the other movable horizontally to simulate the opening and closing of joints or cracks in old pavements beneath an overlay. One limitation of the original work was that long beam samples were required. These were relatively difficult to fabricate in the laboratory and very difficult to get from the field. To solve these problems, an OT was developed with the goal of being able to test 6 in. (150 mm) diameter samples that could be easily fabricated in the lab using a gyratory compactor or obtained from standard field cores (Zhou and Scullion 2005). The upgraded OT is a fully computer-controlled system with special programs. The test data including time, displacement, and force, are automatically recorded and saved as an Excel file. The sample size has been reduced to 6 in (150 mm) long by 3 in (75 mm) wide by 1.5 to 2 in (38 to 50 mm) high, making the OT more practical and easier to handle samples from the SGC or field cores. Figure 2.11 and Figure 2.12 show the schematic diagram and photos of the new TTI OT, respectively.

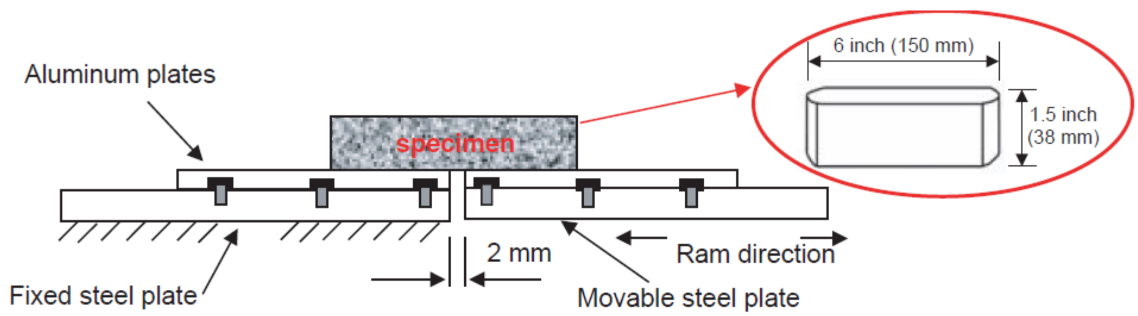



**Figure 2.11. Schematic of Upgraded TTI OT.**

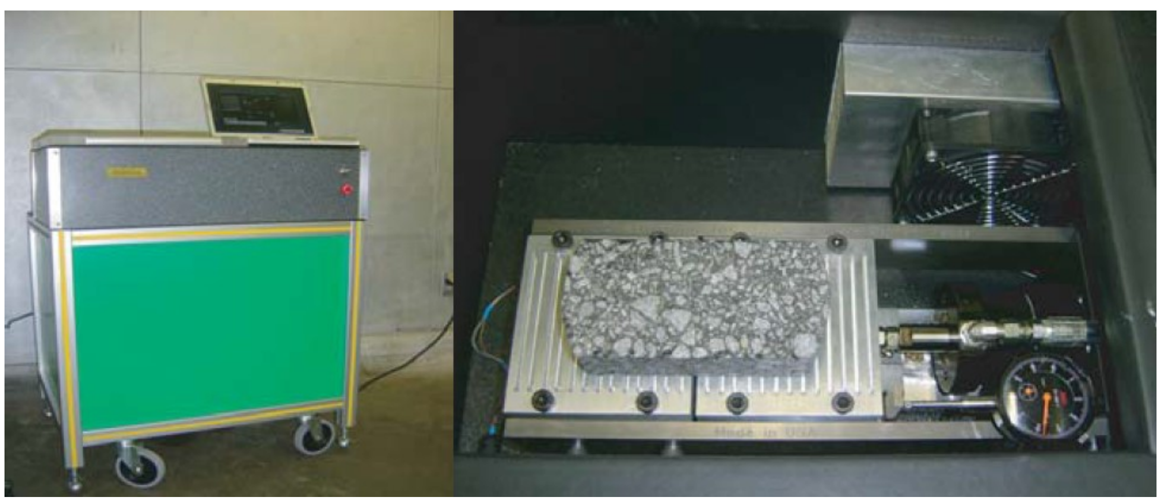


**Figure 2.12. Photos of the OT.**

The Texas Department of Transportation (TxDOT) test procedure Tex-248-F has been in effect since 2009, and a revised version has been in use since 2014. The ASTM version of the test is in process of standardization. The OT test can be conducted in controlled displacement mode under the following conditions:

- Temperature: 0–35 ° C.
- Opening displacement: 0–2 mm.

- Loading rate: 24 hours (or more) per cycle – 10 seconds per cycle.
- Loading type: the loading is applied in a cyclic triangular waveform with constant magnitude.

Tex-248-F requires the test to be conducted at a constant temperature of 77 ± 1 ° F (25 ± 0.5 ° C). The sliding block applies tension in a cyclic triangular waveform to a constant maximum displacement of 0.025 in. (0.06 cm) The sliding block reaches the maximum displacement and then returns to its initial position in 10 sec. (one cycle). The system records the load for each cycle. The test runs until a 93 percent reduction of the maximum load occurs when measured from the first opening cycle, and the cycle number is recorded as the number of cycles to failure. If a 93 percent reduction is not reached within 1,000 cycles, the OT will stop the test.

The OT test has been used in TTI and TxDOT laboratories along with laboratories in other states like New Jersey, Alabama, Oklahoma, Massachusetts, and Nevada. Walubita et al. (2012) noted that the repeatability in the OT test results with a coefficient of variation (COV) of around 30 percent, particularly for most dense- and coarse-graded mixes. Consensus was that variability would be experienced with any repeated load cracking tests and should not be compared with monotonic crack tests or compression loading tests (Walubita et al. 2012). From the literature review, most of the repeated cracking tests were found to exhibit higher COV values, on the order of 65 to 172 percent (SHRP, 1994). Walubita et al. (2012) also presented an evaluation of the critical steps of the OT test procedure in an attempt to optimize the repeatability and minimize variability in the test results. In general, the study indicated that the sample drying method, glue quantity, number of sample replicates, air voids, sample age at the time of testing, and temperature variations are some of the key aspects that have impacts on the OT test repeatability and variability. Overall, findings from this study indicate that variability in the OT test results can be minimized if these aspects are improved and/or more clearly specified in the OT test procedure.

The test can easily capture the effects of asphalt binder content, binder type, aggregate gradation, air void, and other mix design properties (Zhou and Scullion 2005, Zhou et al. 2006, Walubita et al. 2012).

The OT results closely relate to crack propagation in the field (Zhou and Scullion 2005, Zhou et al. 2006, Bennert and Ali 2008, Walubita et al. 2012, Hajj et al. 2010). The OT has been used to simulate anticipated Portland cement concrete horizontal slab movement by determining the coefficient of thermal expansion of the Portland cement concrete, the slab length, and an estimate of the daily change in temperature at the bottom of the HMA layer. Bennert et al. (2009) successfully applied this in a project for Massachusetts DOT to identify reasons for premature reflective cracking on I-495.

### 2.3.5 Indirect Tension Asphalt Cracking Test

Zhou et al. (2017) have developed an indirect tension test for asphalt cracking that requires no cutting, no drilling, no gluing, no notching, no instrumentation, minimal temperature conditioning, and minimal testing time. As seen in Figure 2.13, the loading head is a strip conforming to that required for a simple indirect tension (IDT) test. The only instrumentation required is a load cell and displacement transducer to monitor the force and movement of the loading cross-head, as shown in Figure 2.14. The sample is a

Superpave gyratory specimen compacted or trimmed to a height of 2.42 in (61.5 mm). As developed, the test is performed at 77 ° F (25 ° C), and the repeatability is good with a COV of less than 25 percent.

Figure 2.13. Test Set-Up for Indirect Tension Asphalt Cracking Test (IDEAL-CT).

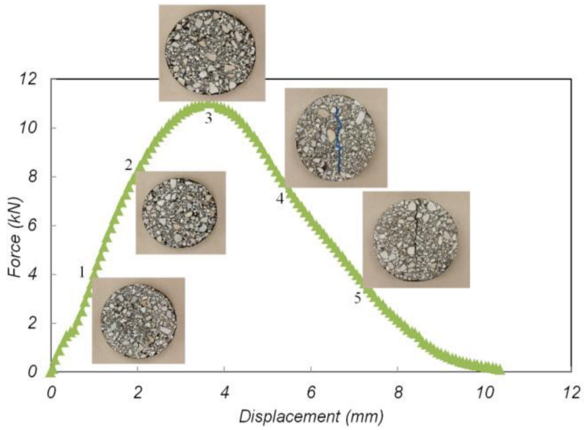



Figure 2.14. Force-Displacement (FD) Curve for IDEAL-CT.

What makes the IDEAL-CT different from a typical IDT test is that the FD curve is used to analyze the results rather than only the maximum stress. In the IDEAL-CT, the load and displacement are monitored and recorded until the complete failure of the sample. Zhou et al. (2017) presented the derivation of the fracture mechanics principles in developing the $CT_{Index}$, which is a function of sample thickness, $G_f$, displacement at 75 percent of the peak load, FD slope at 75 percent of the peak load, and sample diameter.

Some advantages and disadvantages of the IDEAL-CT test are as following:

- **Specimen preparation**: Loose mix is held for four hours at 135 ° C prior to compaction. Sample preparation for the IDEAL-CT test is very simple, requiring only that the mix be laboratory compacted to 7±0.5 percent air voids.
- **Specimen Instrumentation:** None.
- **Testing and Technician Training**: The testing is very rapid as the loading rate is 2 in/min (50 mm/min.) and there is no special temperature conditioning needed as the test is performed

at 77 ° F (25 ° C). The technician training required is the same as the IDT test performed for AASHTO T283.

- **Data Analysis and Result Interpretation**: The analysis simply requires computing the area under the FD curve to determine the fracture energy and other parameters. An Excel spreadsheet template is available to automatically calculate the $CT_{index}$. The larger the $CT_{index}$, the better the cracking resistance.

Zhou et al. (2017) demonstrated that the test on laboratory prepared samples was sensitive to RAP and RAS content (Figure 2.15), binder grade, binder content (Figure 2.16), aging conditions, and air void content. The coefficient of variability is low for a cracking test coming in at an average of 12.7 percent for 15 sets of 3 samples with a high of 23.5 percent and a low of 1.7 percent. This test was recently developed under a NCHRP Idea project and has not yet been used to characterize laboratory mixtures placed in the field. On the whole, this may be the simplest test of the three to implement.

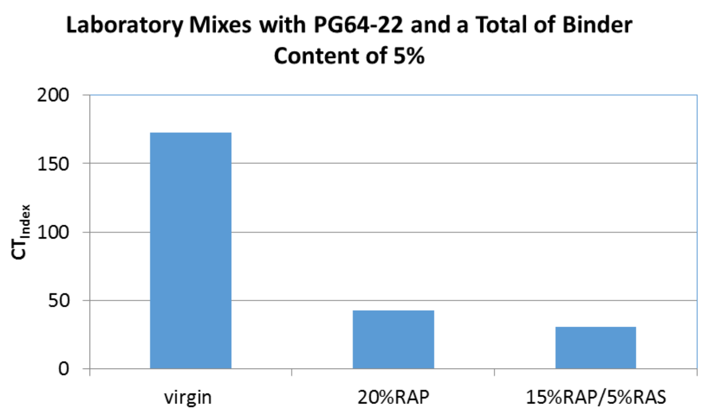



Figure 2.15. IDEAL-CT Sensitivity to RAP and RAS Content.

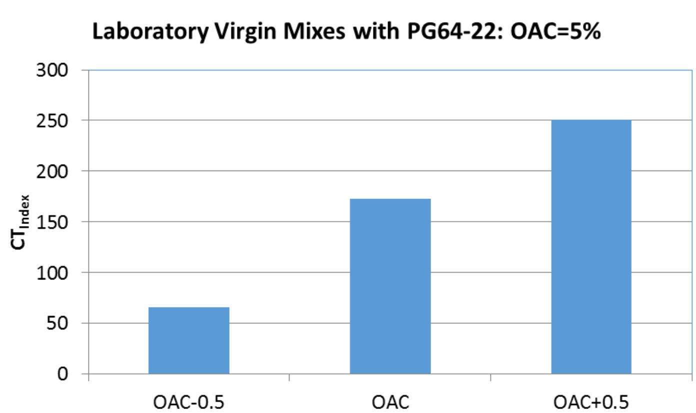



Figure 2.16. IDEAL-CT Sensitivity to Asphalt Content.

## 2.4 RUT TESTING

### 2.4.1 Asphalt Pavement Analyzer

APA is one of three candidate laboratory tests to measure rutting performance of asphalt mixtures. The standard procedure is found in AASHTO T 340: Standard Method of Test for Determining Rutting Susceptibility of Hot Mix Asphalt (HMA) Using the Asphalt Pavement Analyzer (APA). The APA is a second-generation device that was originally developed in the mid-1980s as the Georgia Loaded Wheel Tester; a device designed for rut resistance testing and field quality control. The APA tracks a loaded aluminum wheel back and forth across a pressurized linear hose over a compacted specimen. Commonly used test criteria are 10,000 load cycles using a 100 lb (445 N) load and a 100 psi (690 kPa) hose pressure. Six cylindrical samples 6 in. (150 mm) in diameter by 3 in. (75 mm) tall are required according to AASHTO T 340. The equipment shown in Figure 2.17 includes a chamber for testing at elevated temperatures. Therefore, the APA provides a way to evaluate rut depth without the presence of water. The average rut depth at the end of 8000 cycles is reported.

Kandhal and Cooley (2003b) evaluated the correlation of APA with field rutting performance of asphalt mixtures under NCHRP 9-17, *Accelerated Laboratory Rutting Tests: Evaluation of the Asphalt Pavement Analyzer*. It was concluded that laboratory rut depths measured by the APA had good correlations on individual projects with the field rut depths for the FHWA-accelerated load facility (FHWA-ALF), WesTrack, MnROAD, and I-80 (Nevada) projects. However, the APA-measured rut depths had a poor correlation with field rut depths in the case of 10 test sections on the National Center for Asphalt Technology Test Track, which did not develop any significant rutting after two years of loading. Based on limited data, the APA compared well with other performance tests for predicting the potential for rutting in the field. However, it is generally not possible to predict field rut depths from APA rut depths on a specific project using relationships developed on other projects with different geographical locations and traffic (Kandhal and Cooley 2003b).

**Figure 2.17. APA; a) Equipment Overview, (b) Close-Up View of the Loaded Specimen (Mahoney et al. 2011).**

### 2.4.2 Hamburg Wheel Track Test

The HWTT, originally developed in Germany, is another candidate test for evaluating rutting resistance of asphalt mixtures. The HWTT is often conducted following AASHTO T324: Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA). Both slab specimens and cylindrical specimens can be used. However, cylindrical specimens are often preferred due to easy preparation of SGC specimens (Figure 2.18). Typically two SGC compacted specimens with a diameter of 6 in. (150 mm) and a thickness of 2.5 in. (61 mm) are placed side by side, submerged in water at a temperature between 104 ° F (40 ° C) to 140 ° F (60 ° C), and subjected to 52 passes of a steel wheel of 158 lb (705 N) per minute. Each set of specimens is continuously loaded up to a certain number of load cycles or until the center of the specimen deforms by a specified value.

**Figure 2.18. HWTT.**

A typical HWTT curve in terms of average rut depth versus load cycles can be divided into three main phases: post-compaction, creep, and stripping (Solaimanian et al. 2003). The post-compaction phase refers to the initial consolidation of the specimen. The deformation in the creep phase is primarily a result of the viscous flow of the mixture. The stripping phase starts once the bond between the asphalt binder and the aggregate starts degrading, causing visible damage such as stripping or raveling with additional load cycles. The stripping inflection point (SIP) represents the number of load cycles at which a sudden increase in rut depth occurs, and it is graphically represented as the intersection of the fitted lines that characterize the creep phase and the stripping phase. Currently, the SIP and rut depth at a certain number of load cycles are the main parameters used to evaluate moisture sensitivity and rutting resistance of mixtures, respectively. Mixtures with higher SIP values and lower rut depths are considered to have better performance.

The HWTT has been found to have an excellent correlation with field performance (especially in moisture damage evaluation) (Aschenbrener 1995, Izzo and Tahmoressi 1999, Williams and Prowell 1999). However, it can fail to differentiate between some good and poor performing mixtures (Zhou et al. 2003). HWTT has been widely used by highway agencies, such as California, Colorado, Illinois, Iowa, Louisiana, Montana, Oklahoma, Texas, Utah, Washington, and Wisconsin (Mohammad et al. 2015). There are differences between commercially available HWTT machines in the U.S. market. Direct comparison of HWTT results among different machines may not be appropriate.

### 2.4.3 Flow Number

The FN test was originally developed by the NCHRP 9-19 research team (Witczak et al. 2002) as a simple performance test for evaluating rutting resistance of asphalt mixtures. Since then, it was further refined under NCHRP 9-29 (Bonaquist 2008), and finally was standardized into AASHTO provisional procedure: TP79 *Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)*. Like the dynamic modulus test, the FN specimen is 4 in. (100 mm) diameter by 6 in. (150 mm) tall and is cored from a lab compacted sample of 6 in. (150 mm) diameter by 6.5 in. (170 mm) tall (Figure 2.19). In this test, the FN specimen is subjected to a repeated compressive stress pulse at high temperature. This repeated loading produces permanent strain in the specimen, and the point in the permanent strain curve where the rate of accumulation of permanent strain reaches a minimum value has been defined as the FN. As the FN increases, rutting resistance also increase. It was reported that FN showed good correlation with rutting performance of mixtures from WesTrack, MnROAD, and the FHWA-ALF (Kaloush 2001, Witczak 2007). Also FN criteria were proposed for both HMA and WMA under NCHRP 9-33 (AAT 2011).


**Figure 2.19. Asphalt Mixture Performance Tester for FN Test.**

## 2.5 MOISTURE SUSCEPTIBILITY

Two laboratory tests have received acceptance in United States to evaluate the moisture sensitivity of HMA: the Lottman procedure (AASHTO T 283) and the HWTT (AASHTO T 324). In many cases, the two tests provide different results, likely because they simulate different moisture damage processes. Efforts have been made to improve moisture sensitivity testing using the Environmental Conditioning System developed during the Strategic Highway Research Program (Solaimanian et al. 2003), but these have not yet resulted in a standard test method used by state agencies in the routine design of HMA. The most recent development for conditioning the specimen is the Moisture Induced Stress Tester (MiST$^{TM}$) and its associated standard ASTM D7870: Standard Practice for Moisture Conditioning Compacted Asphalt Mixture Specimens by Using Hydrostatic Pore Pressure.

In AASHTO T 283, 6–8 specimens are divided in two subsets: 3–4 specimens to be tested without conditioning (i.e., dry), and 3–4 specimens to be tested after moisture conditioning. The IDT strength test is performed at room temperature (77 ° F [25 ° C]) under a monotonic load applied at a rate of 2 in./min (50 mm/min). The peak load and specimen dimensions are used to estimate the IDT strength. The ratio of the average tensile strength of the conditioned to unconditioned sample subsets and a visual assessment of stripping are used to measure moisture sensitivity. A mixture is considered to have an acceptable level of moisture sensitivity if the tensile strength ratio is equal to or greater than 80 percent and there is no visual evidence of stripping in the conditioned test specimens.

Since the HWTT (AASHTO T 324) tests HMA submerged in water, it can also be used to evaluate the resistance of a mixture to moisture damage. Moisture sensitivity is evaluated by computing the SIP, which is defined as the intersection of the slopes from the creep and stripping portions of the rut depth versus wheel pass curve. The recommended air void content of laboratory-prepared specimens for AASHTO T 324 is 7.0 ± 1.0 percent. Criteria for evaluating moisture sensitivity based on AASHTO T 324 place a minimum limit on the SIP. For example, Aschenbrener (1995) suggested for Colorado conditions that mixtures with good performance for moisture damage (life of 10 to 15 years) should have an SIP greater than 14,000 passes.

Although the HWTT has been widely used by highway agencies, several issues remain concerning the testing procedure and data analysis. The latest work done under NCHRP 9-49 showed that the current test parameters of SIP and rut depth are not always able to accurately evaluate certain mixtures (Yin et al. 2014). To better analyze the HWTT results, a novel method was developed by Yin et al. (2014), which was able to evaluate moisture sensitivity and rutting resistance separately and with significantly improved accuracy. As shown in Figure 2.20, the HWTT results in terms of rut depth versus load cycle are first fitted by a complex function composed of one part with negative curvature followed by another part with positive curvature. The critical point where the curvature changes is referred to as the Stripping Number (SN), and the load cycle where SN occurs ($LC_{SN}$) is proposed as a parameter to evaluate moisture sensitivity. Then, the Tseng-Lytton model (1989) is employed to fit the viscoplastic strain before stripping, and the slope at the SN ($\Delta\varepsilon_{SN}^{vp}$) is proposed as a rutting resistance parameter. After the SN occurs, the permanent strain induced by stripping (stripping strain) is determined as the difference between the total permanent strain and the projected viscoplastic strain. An exponential model is then used to fit the stripping strain, and the number of additional load cycles after SN needed to reach a total rut depth of 0.5 in (12.5 mm), $LC_{ST}$, the proposed parameter to evaluate moisture sensitivity after stripping. Mixtures with higher $LC_{SN}$ and $LC_{ST}$ values and lower $\Delta\varepsilon_{SN}^{vp}$ values are expected to have better resistance to moisture damage and rutting.

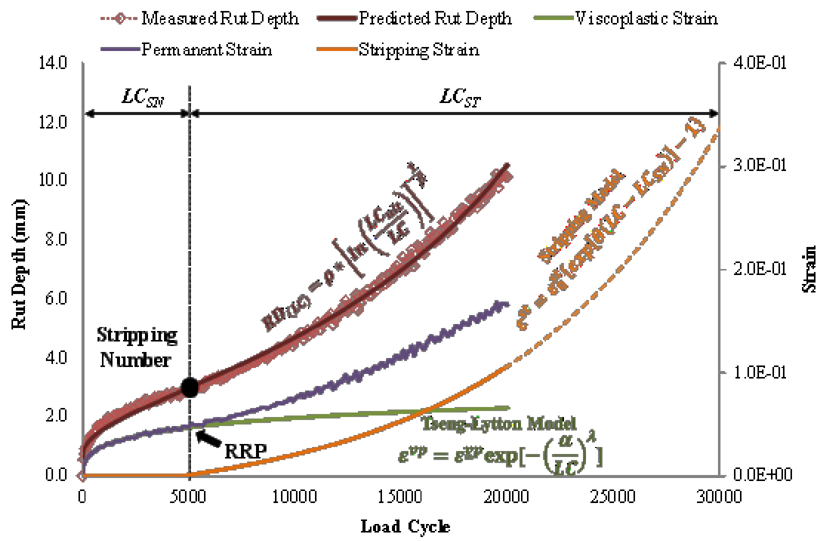



Figure 2.20. HWTT Analysis Methodology (Yin et al. 2014).

## 2.6 STATE-OF-THE-PRACTICE OF BALANCED MIX DESIGN

A number of DOTs have begun to either explore or adopt BMD approaches and others are in the process of investigating performance testing (specifically cracking tests) for integration into their mixture designs. The efforts noted below are those identified by the Task Force as being focused on BMD.

### 2.6.1 California

The California Department of Transportation (Caltrans) uses the performance-modified volumetric design (Aschenbrener 2016). Minimum aggregate quality is specified, as is an aggregate gradation band, along with the asphalt binder grade. Mixture designs are used to establish the initial asphalt binder content. Performance testing consists of repeated shear (AASHTO T 320 "Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester (SST))" and bending beam fatigue test (AASHTO T 321 "Determining the Fatigue Life of Compacted Asphalt Mixtures Subjected to Repeated Flexural Bending"), including frequency sweep testing and HWTT. A short-term conditioning protocol is used for repeated shear and HWTT. Long-term conditioning is used for the bending beam fatigue and frequency sweep. Adjustments to asphalt content are made based on the performance testing results. Other mix design adjustments include binder source, aggregate source, or amount of material passing the No. 200 sieve. A guide to mixture adjustments is being developed. After making these adjustments to meet the performance testing criteria, the mixture is not required to meet the original volumetric criteria.

To date, seven interstate highway projects have been built using this approach. Caltrans is focusing on the mixtures being used on very high-volume pavements.

### 2.6.2 Illinois

The Illinois Department of Transportation (IDOT) has recently begun to use performance testing in addition to volumetric mixture design. The requirements for aggregate quality, gradation, binder grade, and binder quantity have not changed. Asphalt binder content is set using Superpave volumetric mixture design. HWTT with short-term conditioning is used for a rut test. The I-FIT, AASHTO TP124 "Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend (SCB) Geometry at Intermediate Temperature," is used for a cracking test. To achieve the desired performance the asphalt binder content can be increased, the asphalt binder source can be changed or quantities of recycled materials can be reduced. Final volumetric properties are required to be within the Superpave volumetric mixture design criteria.

Approximately 22 projects using this approach were slated for construction in 2016. The primary goal is to address the use of high recycle contents for RAP and RAS.

### 2.6.3 Louisiana

The Louisiana Department of Transportation and Development (LADOTD) recently began to use the volumetric design plus performance testing approach. The Superpave system is used to define the optimum asphalt binder content. The HWTT is done on short-term conditioned specimens to evaluate rutting. Testing for cracking is done using ASTM D 8044-16 "Standard Test Method for Evaluation of Asphalt Mixture Cracking Resistance using the Semi-Circular Bend Test (SCB) at Intermediate Temperatures."

This approach has been implemented in the 2016 LADOTD asphalt specifications. LADOTD is using this mixture design approach for both high- and low-volume roadways on both wearing and binder courses.

### 2.6.4 New Jersey

The New Jersey Department of Transportation (NJDOT) currently uses a procedure based upon volumetric design with performance verification. AASHTO T 340 "Standard Method of Test for Determining Rutting Susceptibility of Hot Mix Asphalt (HMA) Using the Asphalt Pavement Analyzer (APA)" is used for rutting evaluation. For cracking, both the OT and the bending beam fatigue test are used. Short-term conditioning precedes the rutting test and long-term conditioning is applied before the cracking evaluation. Mixture design adjustments include the incorporation of WMA technology, rejuvenators, polymers, and asphalt binder content.

NJDOT uses this approach for about 5 to 10 percent of the state's total asphalt tonnage on mixtures applied to high-volume surfaces (specialty mixtures). Further, Rutgers University has developed a proposed method for the state based entirely upon performance properties, but it has not been used to date.

### 2.6.5 Texas

TxDOT uses volumetric design with performance verification. The Superpave system is used to define the optimum asphalt binder content. Short-term conditioning is used prior to HWTT for rutting evaluation and long-term conditioning is used prior to the OT procedure for cracking evaluation. Mixtures that fail to pass performance testing criteria require a new volumetric design. Mixture design adjustments might include asphalt binder content, binder source, changes in the amount of material passing the No. 200 sieve, or aggregate source.

TxDOT has applied this BMD procedure to specialty mixtures (stone matrix asphalt and thin overlay mixtures) for high-volume surfacing since 2013. Current efforts are investigating mixture design criteria based on climate, pavement substrates, and traffic levels.

### 2.6.6 Wisconsin

The Wisconsin Department of Transportation (WisDOT) is proposing the use of volumetric design with performance testing verification. HWTT after short-term conditioning on the mixture is used for assessing rutting potential. For cracking, both ASTM D7313 "Standard Test Method for Determining Fracture Energy of Asphalt-Aggregate Mixtures Using the Disk-Shaped Compact Tension (DCT) Geometry" and the low-temperature SCB test for cracking evaluation are used. For mixture design adjustments, WisDOT typically allows changes to asphalt binder source, additives, aggregate gradation, or incorporation of rubber.

In 2015, four projects were completed using the BMD method. WisDOT, like IDOT, is using the BMD procedure to address mixtures with high recycled materials content.

## 2.7 SUMMARY

A balanced design for asphalt mixtures consists of performance tests being used to establish a range of binder contents for a given aggregate source and gradation that will avoid rutting and cracking failures and provide good durability. The use of BMD procedures has been identified as a high priority nationally and by several state DOTs as a means of reducing the risks of early failures for asphalt pavements. This literature review has presented a brief history of asphalt mixture design, the findings of the Balanced Mix Design Task Force of the FHWA Mixture Expert Task Group, NCHRP Synthesis No. 492 (McCarthy et al. 2016), performance tests that are suited for implementation in mix design in Minnesota, and a number of states' practices. The BMD Task Force identified three approaches. The BMD approach is currently being used by state DOTs to address high RBR or as a means of ensuring the performance of surface mixtures on high volume roadways. While rutting tests such as the APA, HWTT, and FN are fairly well established, cracking tests are still being evaluated for use in mix design and acceptance. Three cracking tests that seem to relate to thermal cracking (DCT, SCB [Minnesota and Illinois], and OT) have been reviewed. States that have some form of BMD procedure include California, Illinois, Louisiana, New Jersey, Texas, and Wisconsin. The features of these BMD methods have been presented in this literature review. The information contained herein will be used to make recommendations on a BMD procedure for Minnesota.

Based upon the information presented in this chapter, the authors believe the best approach to BMD for Minnesota is to use the volumetric approach to identify an asphalt content that will serve as a starting point. Asphalt mixtures will then be prepared at the volumetric target asphalt content and at

±0.5 percent. These samples will be tested for resistance to rutting and cracking. The target asphalt content for BMD will be: 1) that which passes the cracking criterion plus 0.4 percent to ensure that the asphalt content does not fall below the cracking criterion during production and 2) that which passes the rutting criterion. In the event that the minimum cracking criterion is met at all asphalt contents, the BMD asphalt content will be that which is, at a minimum, 0.4 percent above the cracking criterion. If the material fails one or both criteria, the process would begin again by making adjustments to the job mix formula, i.e., aggregate structure, binder grade, etc.

While it would be a good exercise to try all the performance tests discussed in this chapter, time and funding would not allow it. Thus, based upon the information in this chapter and the authors' previous experience with NCHRP Project 9-57, the following cracking tests were selected for further evaluation: DCT, I-FIT, and IDEAL-CT. These tests are the most relevant in terms of mix design, relationship to performance, and integration into QC/QA processes. For rutting, the HWTT was selected since it is the most ubiquitous among the permanent deformation tests, and has a history of successfully identifying well-performing asphalt mixtures.

# CHAPTER 3: RESEARCH METHODS

## 3.1 EXPERIMENTAL DESIGN AND GOALS

To prove the ability of BMD to provide the required sensitivity to mix design and distinguish between well and bad performing asphalt mixtures, it was necessary to obtain materials from Minnesota that were being used on four actual construction projects. The projects selected had two Nominal Maximum Aggregate Sizes (NMAS) (0.375 and 0.5-in (9.5 and 12.5 mm)) dense gradations and two sources of aggregate. One asphalt source was used in the preparation of the mixture samples to avoid confounding comparisons. The mixtures were prepared using MnDOT's standard mixture design procedure with the compaction level appropriate to the anticipated traffic. The following goals were identified for the experimental design:

1. A comparison of the BMD determined target asphalt content to that obtained from MnDOT's standard volumetric mix design procedure.
2. A comparison between target asphalt contents obtained from the use of the three cracking tests used for evaluation.
3. Illustration of how rutting test results can be used in conjunction with cracking tests.
4. Quantification of the variability of cracking test results.

This chapter presents the characteristics of the materials used in this project, mixture preparation methods, sample preparation, and testing.

## 3.2 MATERIALS

The technical advisory panel and industry partners identified materials for the research team at TTI to use in the development of the BMD. Table 3.1 shows the types and the identified the sources of Minnesota materials to be used in the research. One binder (PG58H-34) and four different types and gradations of aggregates were received. Additional RAP from one source was obtained later in the project to complete the testing.

**Table 3.1. Minnesota Materials Requested and Received Initially.**

| Material | Quantity |
|---|---|
| Binder: PG 58H-34 | 20 gal. (76 l) |
| Aggregate: NMAS 0.5 in (12.5) mm, Carbonate | 500 lb (227 kg) |
| Aggregate: NMAS 0.5 in (12.5 mm), Non-Carbonate | 500 lb(227 kg) |
| Aggregate: NMAS 0.375 in (9.5 mm), Carbonate | 500 lb (227 kg) |
| Aggregate: NMAS 0.375 in (9.5 mm), Non-Carbonate | 500 lb (227 kg) |

MnDOT provided job mix formulas and aggregate combinations (see Table 3.2) for the four mixtures that served as the volumetric designs to provide a baseline against which the BMDs were judged. Thus, the BMD was conducted with the volumetric optimum asphalt content (i.e., at 4 percent air voids) and ±0.5 percent from the volumetric optimum in Task 5.

**Table 3.2. Job Formula for Each Mix.**

| Sieve size (mm) | Mix 1 | Mix 2 | Mix 3 | Mix 4 |
|---|---|---|---|---|
| 19.0 | 100 | 100 | 100 | 100 |
| 12.5 | 94 | 93 | 100 | 100 |
| 9.5 | 85 | 84 | 98 | 98 |
| 4.75 | 69 | 65 | 74 | 73 |
| 2.36 | 54 | 49 | 56 | 54 |
| 1.18 | 43 | 34 | 45 | 38 |
| 0.6 | 30 | 23 | 33 | 25 |
| 0.3 | 14 | 12 | 17 | 12 |
| 0.15 | 6 | 5 | 7 | 5 |
| 0.075 | 3.8 | 2.7 | 4 | 3.1 |
| %AC (PG 58H-34) | 5.4 | 5.4 | 5.8 | 6.0 |
| Aggregate combinations | Sand: 27%<br>3/8" (9.5 mm) chip: 7%<br>Lime sand superpave: 21%<br>3/4" (19 mm) clear: 20%<br>RAP: 25% | CA-50: 22%<br>1/2" (12.5 mm) clear: 11%<br>Washed sand1: 25%<br>Washed sand2: 14%<br>#2 screened sand: 8%<br>RAP: 20% | Sand: 30%<br>3/8" (9.5 mm) chip: 27%<br>Lime sand superpave: 23%<br>RAP: 20% | 1/2" (12.5 mm) clear: 28%<br>Washed sand1: 23%<br>Washed sand2: 15%<br>#2 screened sand: 14%<br>RAP: 20% |
| Traffic Level | 3 | 4 | 3 | 4 |
| No. Gyrations | 60 | 90 | 60 | 90 |

As might be expected, the nominal maximum aggregate size and gradation defined the volumetric optimum asphalt content more than the type of aggregate or number of gyrations used in compaction. The gradations for the 0.375 in (9.5 mm) NMAS mixtures (Mix 3 and Mix 4) had volumetric optimum asphalt contents that were 0.4 percent and 0.6 percent, respectively, greater than for the 0.5 in (12.5 mm) NMAS mixtures (Mixes 1 and 2). Figure 3.1 through Figure 3.4 show the gradations for the four mixtures. The gradations show that Mixes 1 and 3 have gradations, which are further removed from the line of maximum packing represented by the blue line than Mixes 2 and 4. All mixtures are fine-graded and meet the requirements of MnDOT Specification 2360, “Plant Mixed Asphalt Pavement.”

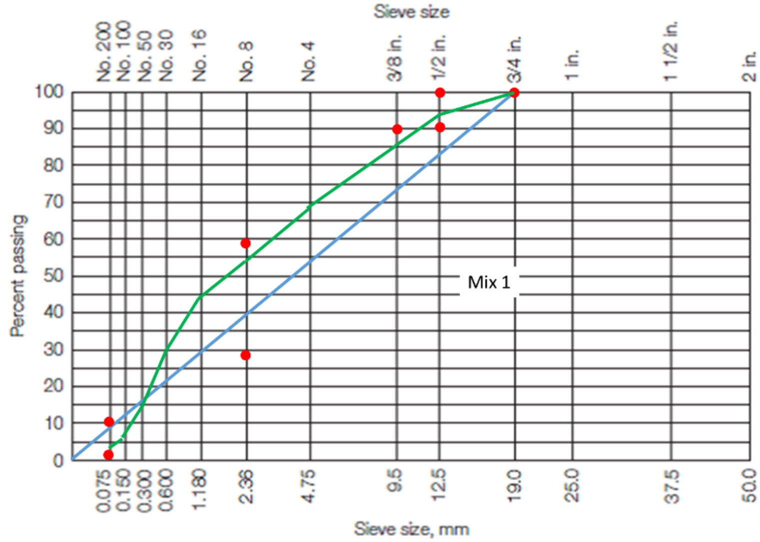



**Figure 3.1. Gradation for Mix 1 (0.5 in (12.5 mm) Superpave, Carbonate, Level 3).**

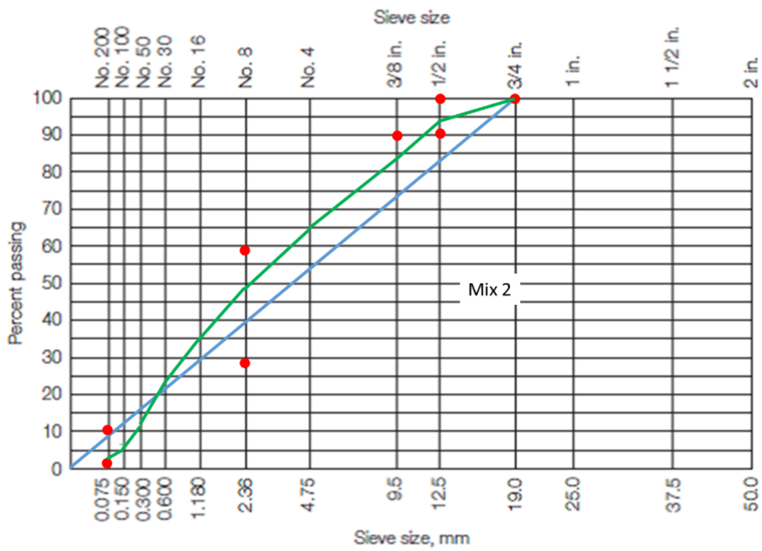



**Figure 3.2. Gradation for Mix 2 (0.5 in (12.5 mm) Superpave, Non-Carbonate, Level 4).**

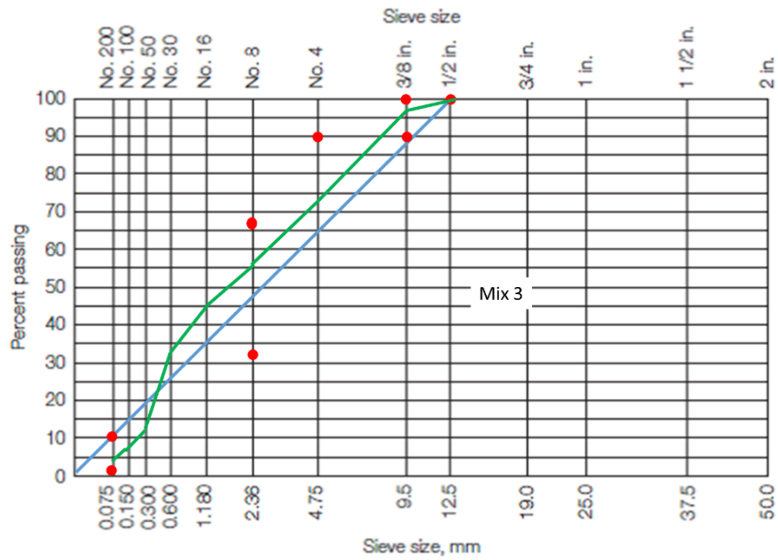



**Figure 3.3. Gradation for Mix 3 (0.375 in (9.5 mm) Superpave, Carbonate, Level 3).**

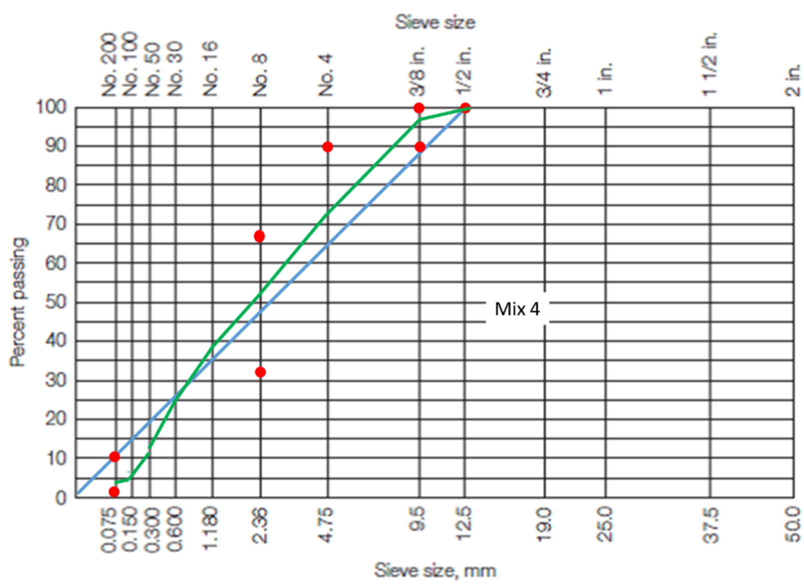
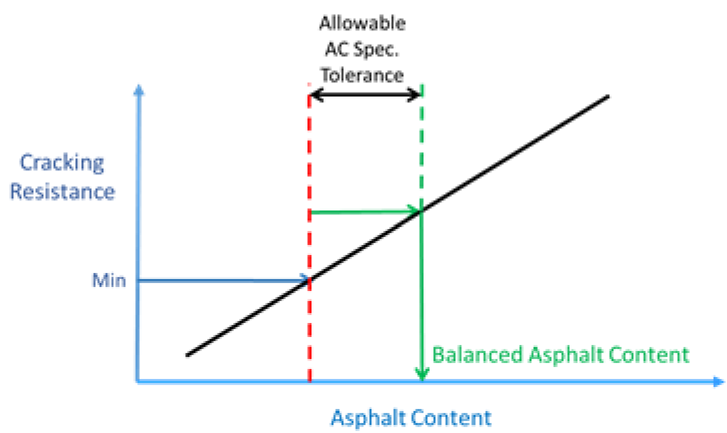



**Figure 3.4. Gradation for Mix 4 (0.375 in (9.5 mm) Superpave, Non- Carbonate, Level 4).**

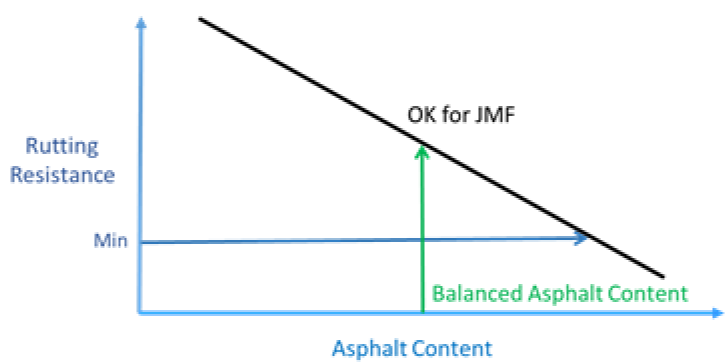
## 3.3 PERFORMANCE TEST METHODS

Performance tests are used in BMD to define the boundaries of acceptable asphalt contents. Since, the trend is for cracking resistance to increase with increasing asphalt content, the minimum level of cracking resistance may be used to establish the minimum required asphalt content. To avoid construction variation that may result in dry mixes, the minimum asphalt content from the cracking test should have the allowable lower specification limit added to it as shown in Figure 3.5(a). The opposite is true for rutting in that rutting resistance increases with decreasing asphalt content. The asphalt content determined from the cracking test is then compared to the maximum allowable asphalt content defined by the results of the rutting test (Figure 3.5(b)). If the value of the design asphalt content from the cracking criterion falls below the maximum asphalt content from the rutting test, then the mixture at that asphalt content becomes the job mix formula. If the window for acceptable asphalt content is too narrow to allow for normal construction tolerance, changing the job mix formula by changing the aggregate gradation or source, or the asphalt grade or source, is recommended.

In this project, three types of cracking test and one rutting test were used to define these boundaries. The cracking tests included DCT test, the Illinois version of the SCB test referred to as the Illinois Fracture Index Test (I-FIT), and the Indirect Tension Asphalt (IDEAL-CT) Test. The DCT is already in use in Minnesota as a low-temperature cracking test. The I-FIT and IDEAL-CT tests are both intermediate temperature cracking tests. The rutting test selected is the HWTT, which is performed at a relatively high temperature. The HWTT was selected for rutting evaluation due to its use by a number of DOTs. These tests were described in detail in Chapter 2.




**(a)**




**(b)**

**Figure 3.5. Allowable Minimum Asphalt Content Considering Construction Variation Using (a) Cracking Criterion and (b) Check on Rutting Criterion.**

The performance criteria selected for this project come from those proposed by the test developers or other agencies that use them. Table 3.3 and Table 3.4 present these criteria and the test conditions.

**Table 3.3. Performance Criteria for Cracking Tests.**

| Test | Test Temperature, ° C | Loading Rate, mm/min | Test Criteria | Source |
|---|---|---|---|---|
| DCT (Low Vol.)<br>DCT (High Vol.) | PG<br>LT+10 ° C | 1 | 450 $J/m^2$<br>500 $J/m^2$ | Wagoner et al. 2006 |
| I-FIT | 25 | 50 | 8 (index value) | Al Qadi et al. 2015 |
| IDEAL-CT | 25 | 50 | 80 (index value) | Zhou et al. 2017 |

**Table 3.4. Wisconsin Performance Criteria for HWTT.**

| Binder Grade | No. of Cycles to 0.5 in (12.5 mm) Rut Depth* |
|---|---|
| 58S | 5,000 |
| 58H | 10,000 |
| 58V | 15,000 |
| 58E | 20,000 |

*Test performed at 45 ° C.

## 3.4 BALANCED MIX DESIGN

Figure 3.6 shows the process of performing a BMD as proposed for MnDOT. The first portion of the method is to select the materials for use according to the current practice. Aggregates should meet the consensus properties and gradation required for the particular application, and the asphalt grade should be selected according to Superpave guidelines found in the LTPPbind software. Materials are combined, mixed, and short-term oven aged (STOA) for 2 hr for the rutting test and long-term oven aged (LTOA) for 4 hr for the cracking test at the suggested compaction temperature. Using a volumetric design, the asphalt content ($AC_v$) meeting the requirement of 4.0 percent air voids at $N_{design}$ is defined. Next, samples are prepared at $AC_v$, $AC_v$ +0.5 percent, and $AC_v$ −0.5 percent. Work is being conducted under NCHRP 9-61 to better define STOA and LTOA protocols for use in sample preparation. After aging, samples should be compacted to 7±0.5 percent air voids. This level of target air voids is used to represent what might be expected in field compaction. The asphalt content defined as the balanced asphalt content ($AC_B$) should be selected according to the test results and accounting for the allowable variance of asphalt content in construction. The addition of the construction tolerance will help ensure that the resulting field mixture does not fall below the minimum required by the cracking performance testing. An example will be presented next.

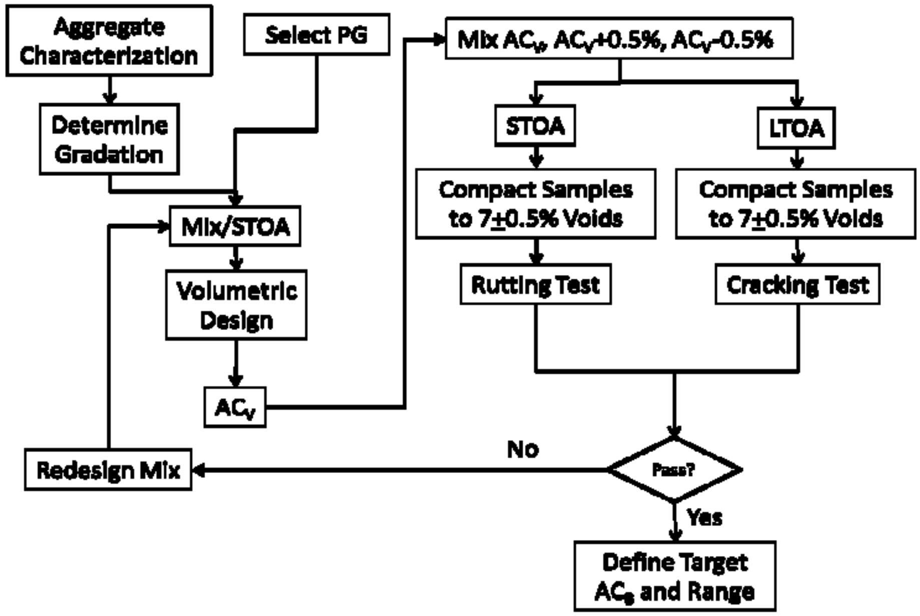



**Figure 3.6. Flow Chart of BMD Approach Using Volumetric Data as a Starting Point.**

The performance testing was used on the mixtures to determine the balanced asphalt content at an air void content of 7±0.5 percent. The performance tests were selected because they all account for post-peak load behavior, which helps differentiate between brittle and ductile behavior.

# CHAPTER 4: RESULTS OF BALANCED MIX DESIGN AND PERFORMANCE TESTING

Using the BMD approach described in Chapter 3, the materials for the four mixtures supplied by MnDOT (Chapter 2) were mixed and compacted in accordance with AASHTO R 35. The $AC_v$ was determined and then performance testing samples were prepared at $AC_v$ and at $AC_v$ + 0.5 percent and $AC_v$ – 0.5 percent. The number of replicates for each test were 6 for I-FIT, 5 for IDEAL-CT, and 5 for DCT. The HWTT was performed on one set of samples to check the acceptable range of asphalt contents. Below are the results of the BMD determined asphalt contents ($AC_B$) compared to the $AC_v$ as well as an analysis of the variability of test results.

## 4.1 BALANCED MIX DESIGN

### 4.1.1 Mixture 1 – 0.5 in (12.5 mm), Carbonate Aggregate, Level 3

Figure 4.1 through Figure 4.4 present the results of performance testing for Mix 1. As can be seen in Figure 2.3 and as described in Chapter 3, the approach to determining the asphalt content for cracking is to take either the asphalt content at the minimum acceptable value for cracking resistance (FI in this case) or the lowest value for cracking resistance if all values meet the required FI (8.0) and add 0.40 percent asphalt to that value. This will help ensure that during production, the lowest asphalt content from production still meets the minimum requirement. This approach applies to all of the cracking tests.

Figure 4.1 shows that the FI increases with asphalt content and that the FI for all three asphalt contents pass the requirement of 8.0. Adding 0.4 percent asphalt above the minimum value to ensure sufficient asphalt content (Figure 3.5, Chapter 3) for the FI shows that the asphalt content for cracking resistance should be 5.3 percent for production.

Figure 4.2 shows the results for the IDEAL-CT test. In this case, the cracking criteria of 80 is only met at asphalt contents higher than 5.5 percent so the asphalt content should be 5.9 percent for production.

Figure 4.3 shows that Mix 1 will not meet the DCT criterion for high volume pavements at any asphalt content within the testing range. Since this is a Level 3 mixture, this would probably be expected. For the low volume DCT criterion, asphalt contents above 5.4 percent would pass the 450 $J/m^2$ criterion. Thus, the production asphalt content should be 5.8 percent, providing the mixture passes the rutting criteria.

The next step is to check the rutting resistance of the mix with the HWTT as shown in Figure 4.4. All the mixes pass the 58H requirement of 0.5in (12.5 mm) rut depth at more than 10,000 cycles (Table 3.4, Chapter 3). The asphalt content for the selected cracking test would then be the initial production asphalt content. However, the rutting results show that the maximum allowable asphalt content is only 6.1 percent, which could be a very challenging asphalt content range for the IDEAL-CT and DCT designed mixtures in production. Given that this is a Level 3 mixture, it is suggested that future work focus on the appropriateness of the performance criteria. The asphalt content to satisfy the volumetric criteria only differs by 0.1 percent from the performance asphalt content by I-FIT, but is 0.4 to 0.5 percent lower according to the IDEAL-CT and DCT test results. It is suggested that the cracking and rutting resistance should be checked at least once after the start of production.

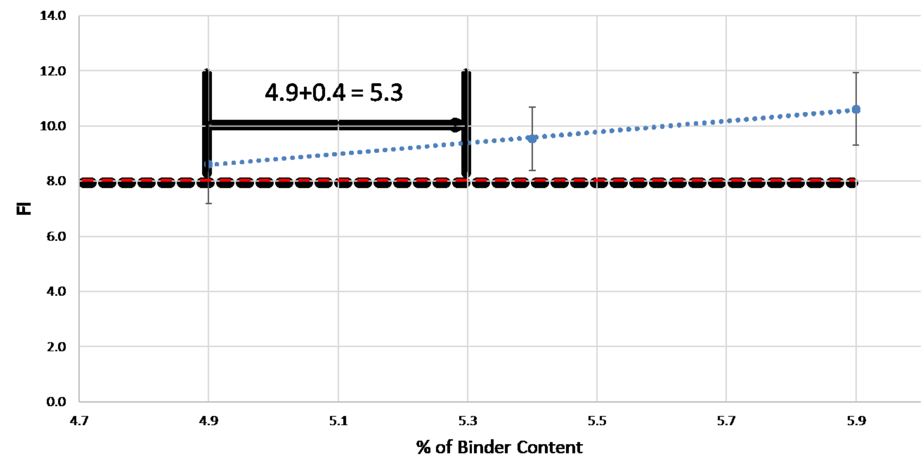



Figure 4.1. I-FIT Test Results for Mix 1.

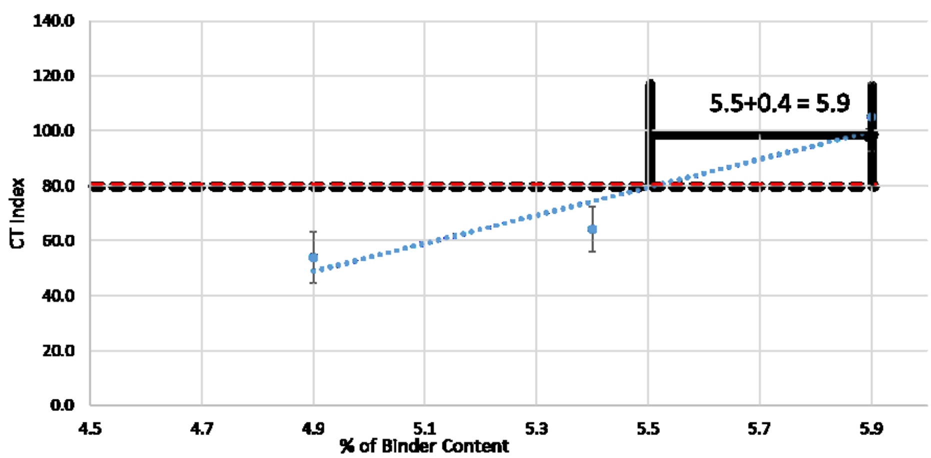



Figure 4.2. IDEAL-CT Test Results for Mix 1.

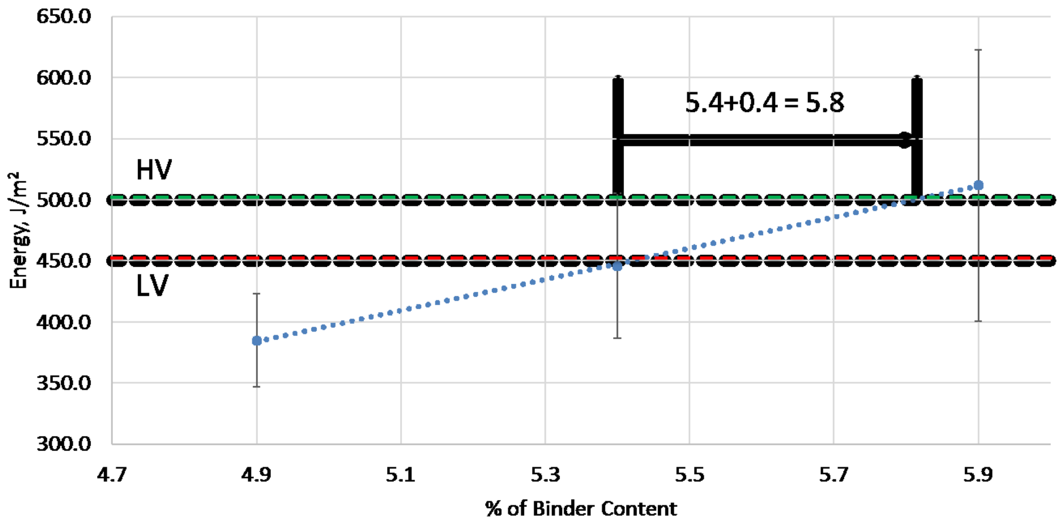



Figure 4.3. DCT Test Results for Mix 1.

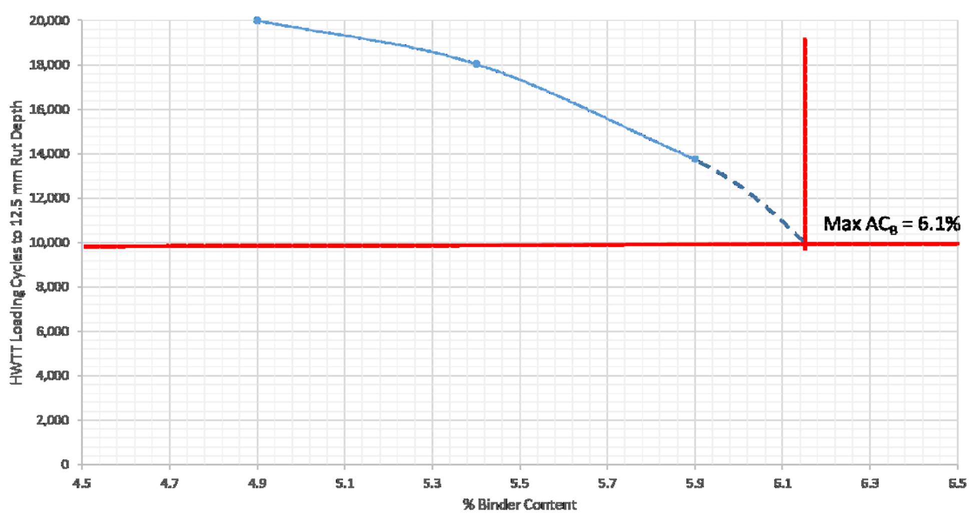



Figure 4.4. HWTT Results for Mix 1.

### 4.1.2 Mixture 2 – 0.5 in (12.5 mm), Non-carbonate Aggregate, Level 4

Figure 4.5 through Figure 4.8 present the results for Mixture 2. This is a 0.5 in (12.5 mm) mix with a non-carbonate aggregate, compacted with 90 gyrations. As can be seen, all three cracking tests produced the same cracking resistance asphalt content of 5.3 percent with the allowance for production tolerance. The lowest asphalt cracking resistance in all three cases occurred just above the minimum criterion. For this mixture, the DCT requirement for high volume roads 500 J/m$^2$ was satisfied. Also, the HWTT criterion was satisfied by all three asphalt contents with all values being above the testing cut-off point of 20,000 cycles. This mixture would be very suitable for a high-volume surface mixture, as it should be for a Level 4 mixture. In this case, the cracking resistance asphalt contents by all three cracking tests are 0.1 percent lower than the volumetric optimum asphalt content.

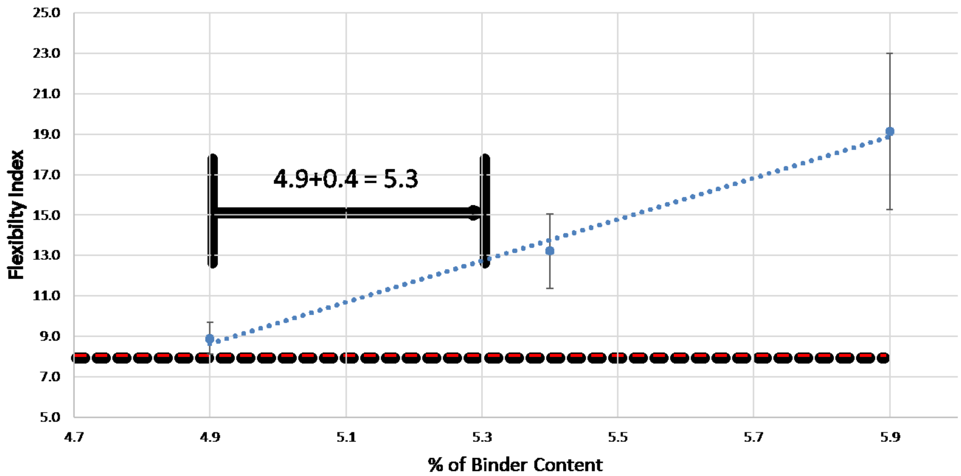



Figure 4.5. I-FIT Results for Mix 2.

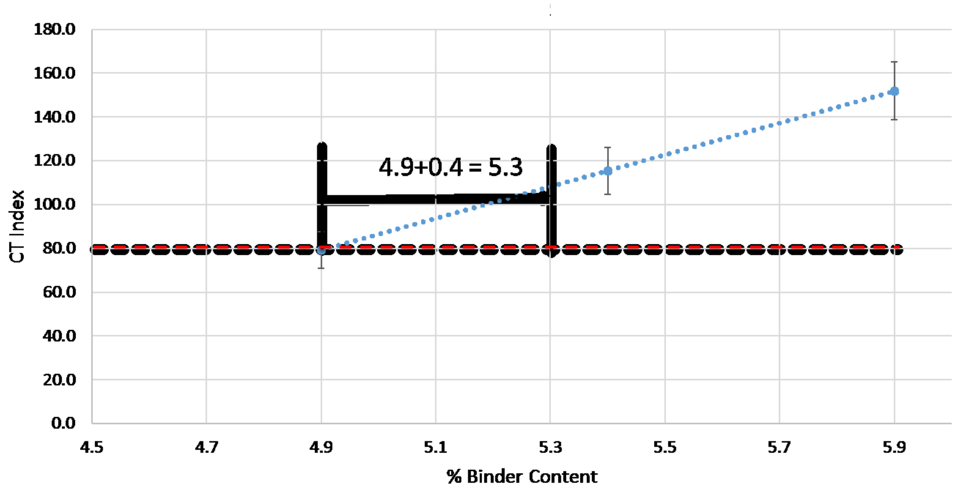



Figure 4.6. IDEAL-CT Results for Mix 2.

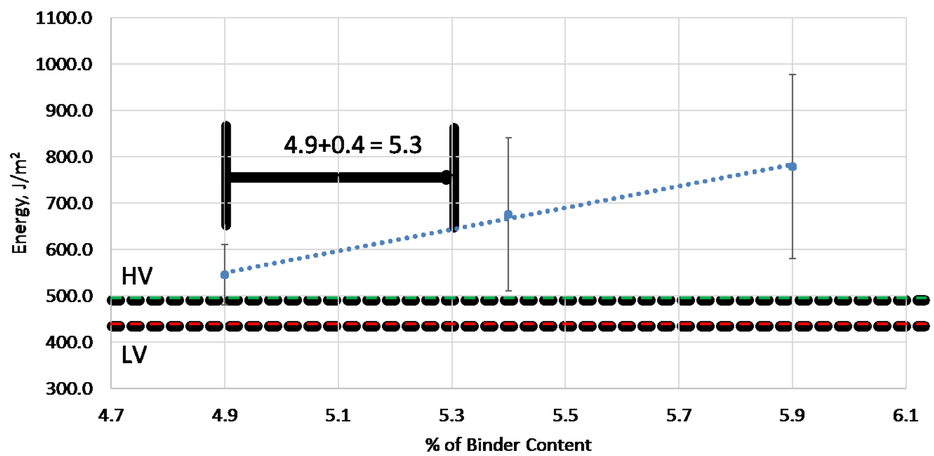



**Figure 4.7. DCT Test Results for Mix 2.**

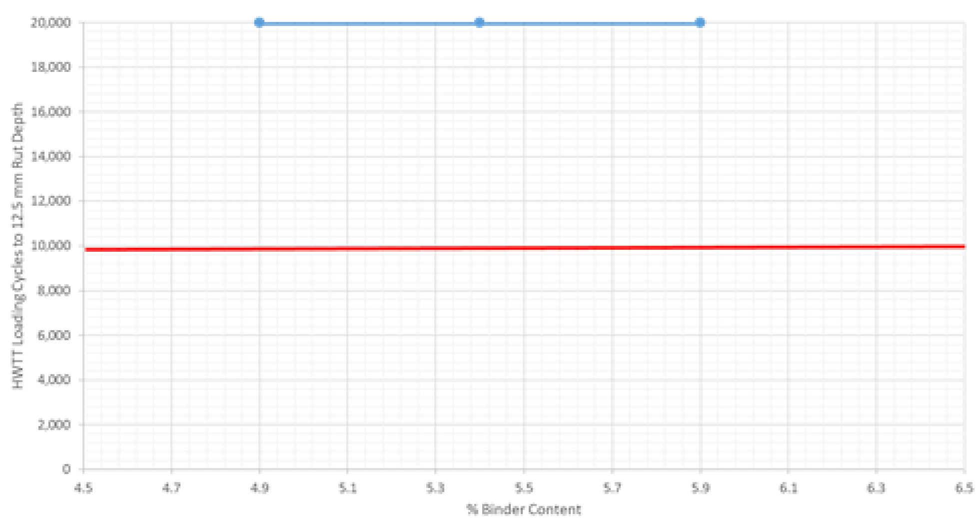



**Figure 4.8. HWTT Results for Mix 2.**

### 4.1.3 Mixture 3 – 0.375 in (9.5 mm), Carbonate Aggregate, Level 3

Mixture 3 was a 0.375 in (9.5 mm) NMAS made with carbonate aggregate and compacted with 60 gyrations of the SGC (Level 3). Figure 4.9 through Figure 4.12 present the results for Mixture 3.

As with the previous carbonate, Level 3 combination (Mix 1), the I-FIT procedure finds that all three asphalt contents pass the minimum criterion of a FI of 8.0. For the I-FIT the cracking resistance asphalt content is 5.7 percent. This is not the case for the IDEAL-CT where the asphalt content needs to be greater than 5.8 percent to pass the criterion of 80. For the IDEAL-CT the asphalt content for production is 6.2 percent. The DCT results are very interesting in that the low volume asphalt content agrees with the I-FIT results and the high volume asphalt content agrees with the IDEAL-CT results. These differences between the cracking tests results could indicate the need to develop low and high volume criteria for the I-FIT and IDEAL-CT tests. However, this observation is based only on the results for one mixture.

The rutting resistance results show that this 0.375 in (9.5 mm) mix is very sensitive to changes in asphalt content. An asphalt content above 6.0 percent could lead to rutting issues during performance. However, if the mix is to be used in a thin layer application, rutting would not necessarily be a problem. Since this is a Level 3 mix, most likely it would be used in a low-volume road, and the performance criteria need to be addressed in future research to allow for a reasonable production variability.

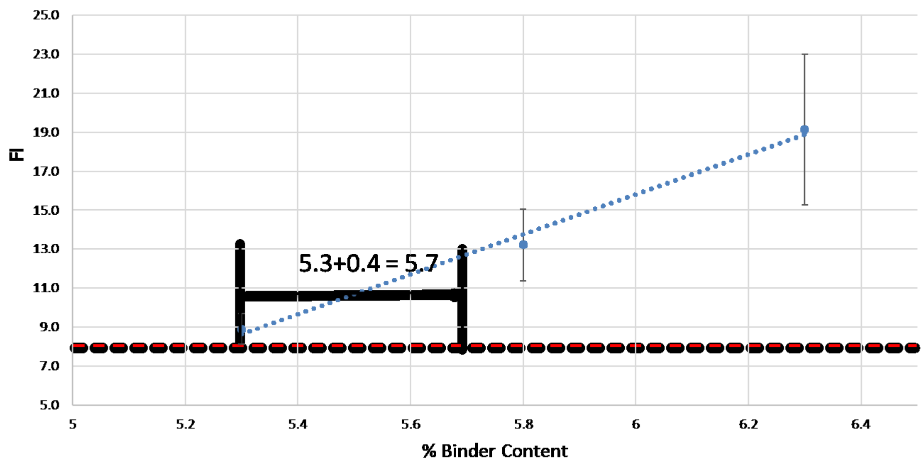



**Figure 4.9. I-FIT Results for Mix 3.**

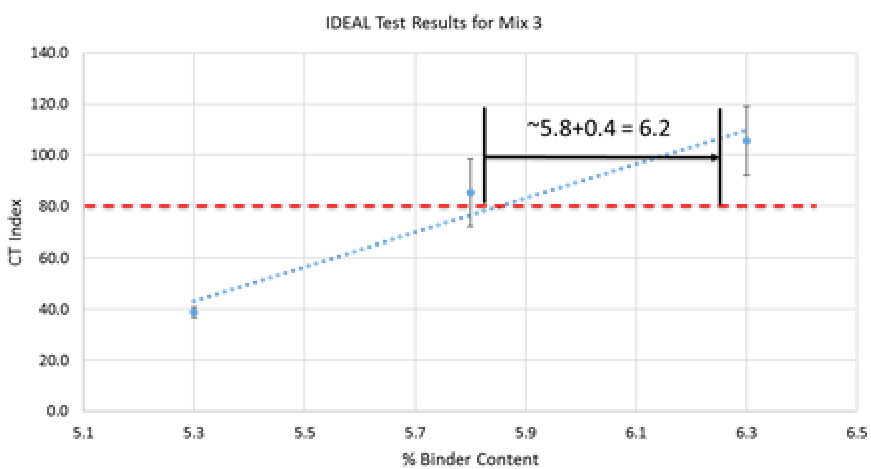



**Figure 4.10. IDEAL-CT Results for Mix 3.**

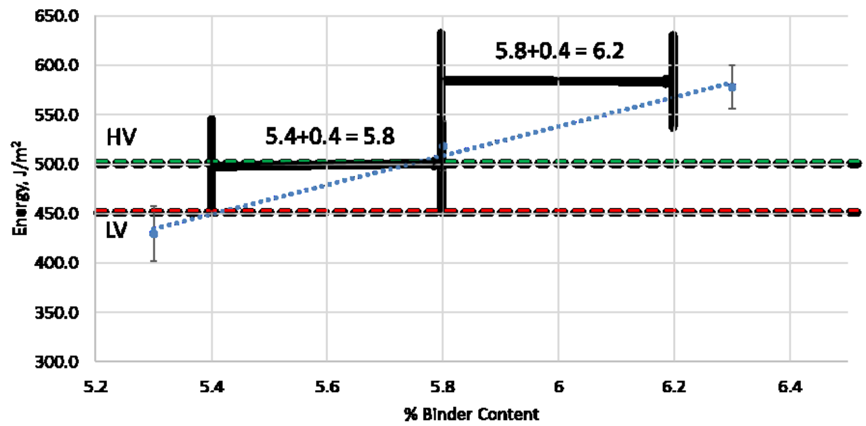



Figure 4.11. DCT Results for Mix 3.

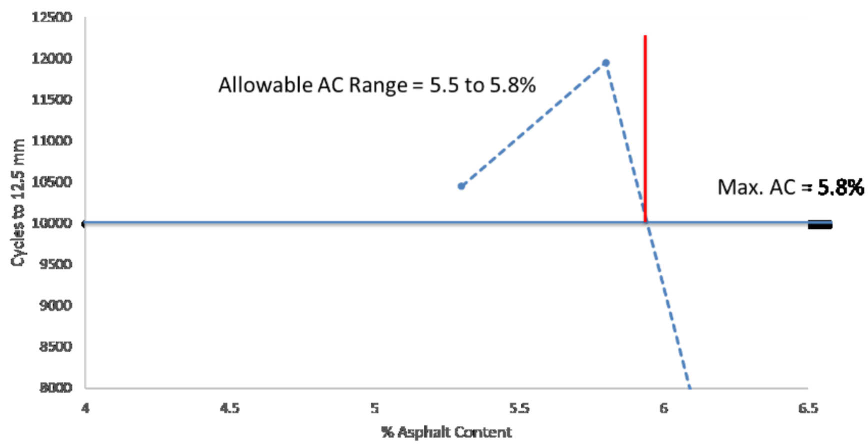



Figure 4.12. HWTT Results for Mix 3.

### 4.1.4 Mixture 4 – 0.375 in (9.5 mm), Non-carbonate Aggregate, Level 4

Mixture 4 was another 0.375 in (9.5 mm) mix produced with a non-carbonate aggregate and 90 gyrations of the SGC. Figure 4.13 through Figure 4.16 show the results. As with Mix 2 (also non-carbonate, Level 4), all the cracking tests provided the same cracking resistance asphalt content. However, due to the smaller aggregate, the asphalt content was higher in this case, 5.9 percent, which is 0.1 percent lower than the volumetric asphalt content. As with the previous 0.375 in (9.5 mm) mixture, the rutting resistance is sensitive to asphalt content with a maximum allowable level of 6.2 percent, but it should perform satisfactorily at the asphalt content of 5.9 percent. Although the I-FIT and DCT results show that these cracking test criteria exceed their proposed minimums at the minimum asphalt content, extrapolation to the asphalt content to the minimum required criteria is not recommended. The cracking test results could deteriorate at an accelerated rate outside the tested range of asphalt contents. It is conceivable that the tests could be performed at lower asphalt contents to find the minimum acceptable value.

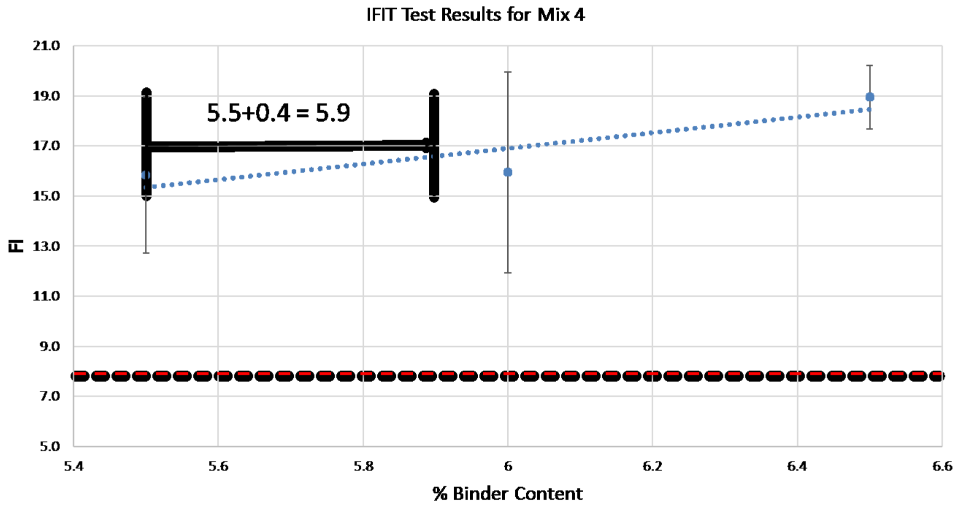



Figure 4.13. I-FIT Results for Mix 4.

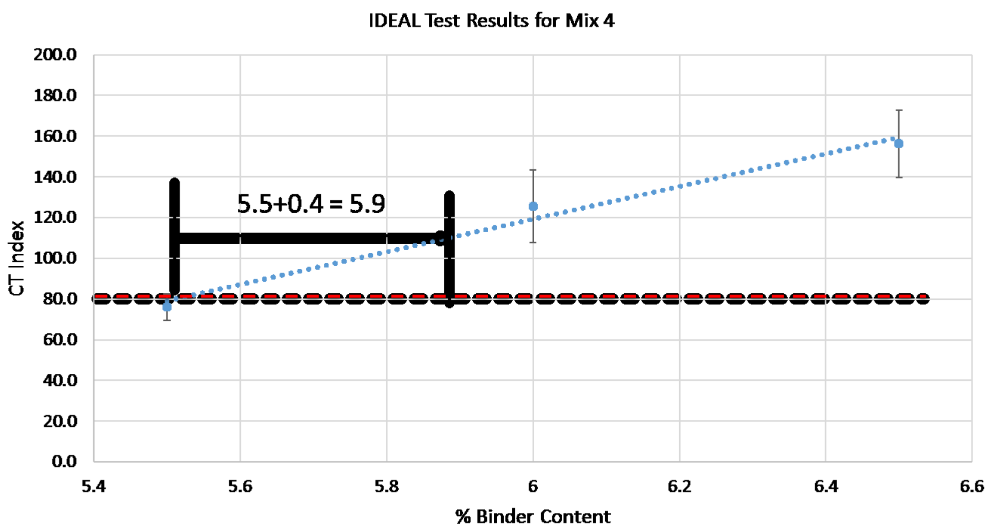



Figure 4.14. IDEAL-CT Results for Mix 4.

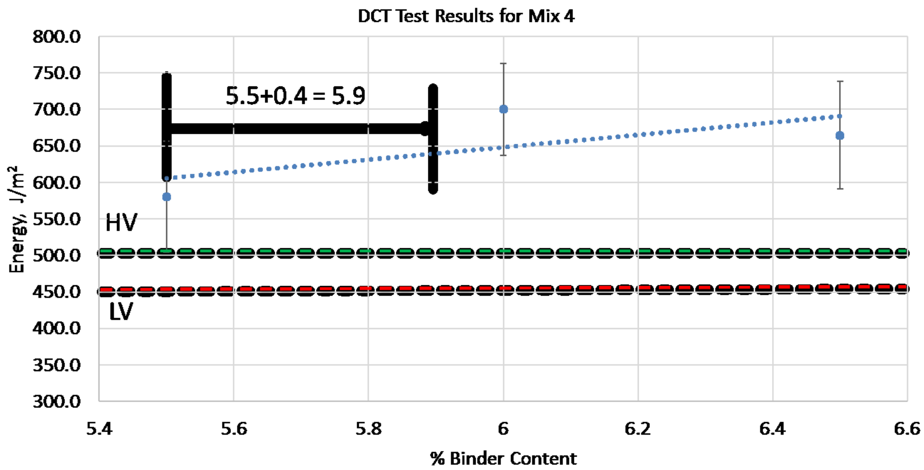



Figure 4.15. DCT Test Results for Mix 4.

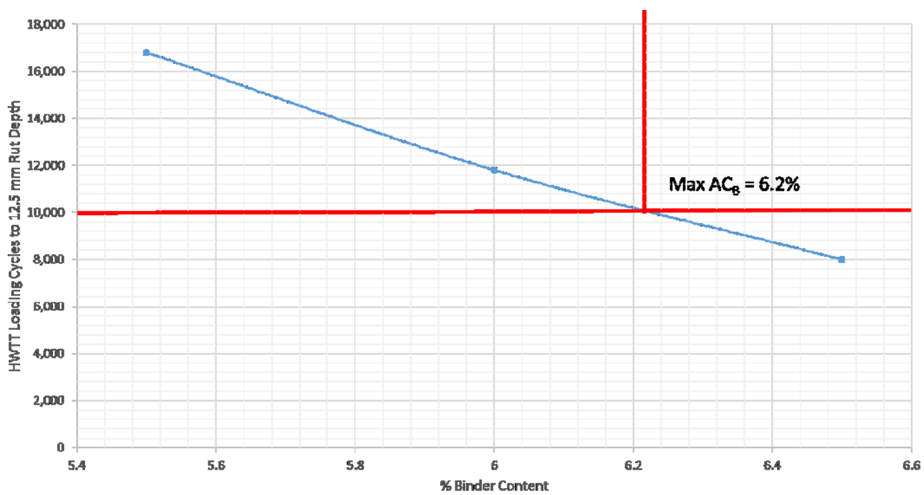



Figure 4.16. HWTT Results for Mix 4.

## 4.2 PERFORMANCE TESTING CHARACTERISTICS

Table 4.1 presents the cracking resistance and variability using the DCT (-11 °F (−24 °C)) (LPG+10), the I-FIT (77 °F (25 °C)), and the IDEAL-CT (77 °F (25 °C)). The optimum asphalt content from the cracking test is also listed according to the results presented in the interim report. Recall that the optimum asphalt content is the lowest asphalt content passing the criterion plus the 0.4 percent construction allowance. There are some things to note in the data. The cracking resistance in almost all circumstances increases with asphalt content, and the optimum asphalt content as determined by the various cracking tests are, for the most part, very close to one another (0.1 to 0.2 percent). The exception to this is Mix 1 in which the I-FIT test gave an asphalt content that was 0.6 percent lower than the IDEAL-CT test and 0.5 percent lower than the DCT test. The average COV is remarkably close for all three tests with 15.75 percent, 12.92 percent, and 11.50 percent for the I-FIT (6 replicates), DCT (5 replicates), and

IDEAL-CT (5 replicates), respectively. These are also very reasonable COVs for laboratory performance testing, noting that they are for a single operator.

Table 4.2 gives the results of the HWTT. Since HWTT is usually performed on one pair of cores in line with each other, there are no variability statistics to report. However, information on the HWTT variability may be found in Harrigan (2014). For a single operator for the number of cycles to 0.5 in (12.5 mm) rut depth, the repeatability COV was approximately 17 percent, which is a little higher than the cracking tests discussed earlier. The multi-laboratory reproducibility COV is about 22 percent. The Wisconsin procedure, which follows AASHTO T 324-11, was used for performing the test. Wisconsin specifies that the test be performed at 113 ° F (45 ° C) and that the sample not exceed a rut depth of 0.5 in (12.5 mm) for a number of cycles determined by the high temperature performance grade as shown in Table 4.3. For the PG 58H-34 binder used in this study, the requirement is 10,000 cycles or above.

**Table 4.1. Summary of Cracking Performance Data.**

| Mix No. | I-FIT | | | | IDEAL-CT | | | | DCT | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **AC, %** | **Mean** | **Std. Dev.** | **COV** | **AC, %** | **Mean** | **Std. Dev.** | **COV** | **AC, %** | **Mean** | **Std. Dev.** | **COV** |
| **1** 12.5mm Carb Level 3 | 4.9 | 8.6 | 1.43 | 17 | 4.9 | 53.9 | 9.28 | 17 | 4.9 | 385.0 | 38.12 | 10 |
| | 5.4 | 9.5 | 1.14 | 12 | 5.4 | 64.2 | 8.11 | 13 | 5.4 | 446.0 | 59.00 | 13 |
| | 5.9 | 10.6 | 1.32 | 12 | 5.9 | 105.0 | 12.44 | 12 | 5.9 | 511.8 | 110.94 | 22 |
| **Opt.** | 5.3 | | | | 5.9 | | | | 5.8 (LV) | | | |
| **2** 12.5mm Non-Carb Level 4 | 4.9 | 8.9 | 0.78 | 9 | 4.9 | 79.4 | 8.32 | 10 | 4.9 | 545.4 | 64.91 | 12 |
| | 5.4 | 13.2 | 1.85 | 14 | 5.4 | 115.3 | 10.83 | 9 | 5.4 | 676.0 | 164.38 | 24 |
| | 5.9 | 19.1 | 3.87 | 20 | 5.9 | 151.9 | 13.3 | 9 | 5.9 | 779.2 | 198.05 | 25 |
| **Opt.** | 5.3 | | | | 5.3 | | | | 5.3 (LV or HV) | | | |
| **3** 9.5mm Carb Level 3 | 5.3 | 6.5 | 1.02 | 16 | 5.3 | 38.8 | 2.05 | 5 | 5.3 | 429.8 | 28.42 | 7 |
| | 5.8 | 11.7 | 3.05 | 26 | 5.8 | 85.3 | 13.24 | 16 | 5.8 | 518.4 | 24.86 | 5 |
| | 6.3 | 18.8 | 2.05 | 11 | 6.3 | 105.5 | 13.53 | 13 | 6.3 | 577.6 | 21.88 | 4 |
| **Opt.** | 5.7 | | | | 6.2 | | | | 5.8 (LV) 6.2 (HV) | | | |
| **4** 9.5mm Non-Carb Level 4 | 5.5 | 15.8 | 3.11 | 20 | 5.5 | 76.0 | 6.47 | 9 | 5.5 | 580.2 | 75.98 | 13 |
| | 6.0 | 15.9 | 4.01 | 25 | 6.0 | 125.4 | 17.82 | 14 | 6.0 | 700.0 | 62.66 | 9 |
| | 6.5 | 19.0 | 1.27 | 7 | 6.5 | 156.3 | 16.52 | 11 | 6.5 | 664.6 | 73.41 | 11 |
| **Opt.** | 5.9 | | | | 5.9 | | | | 5.9 (HV) | | | |
| **Mean** | | | 2.08 | 15.75 | | | 10.99 | 11.50 | | | 76.88 | 12.92 |
| **Range** | | | 0.78–4.01 | 7–26 | | | 2.05–17.82 | 5–17 | | | 21.88–198.05 | 4–25 |

Table 4.2. HWTT Results for Mixtures at 113 ° F (45 ° C).

| Mixture | AC, % | No. Cycles | Rut Depth (mm) |
|---|---|---|---|
| **1 (0.5 in (12.5 mm), Carb., Level 3)** | 4.9 | 20,000* | 5.76 |
| | 5.4 | 18,050 | 12.50 |
| | 5.9 | 13,750 | 12.81 |
| **2 (0.5 in (12.5 mm), Non-Carb., Level 4)** | 4.9 | 20,000* | 6.44 |
| | 5.4 | 20,000* | 6.12 |
| | 5.9 | 20,000* | 8.78 |
| **3 (0.375 in (9.5 mm), Carb., Level 3)** | 5.3 | 10,450 | 12.81 |
| | 5.8 | 11,950 | 12.52 |
| | 6.3 | 5,150 | 12.55 |
| **4 (0.375 in (9.5 mm), Non-Carb., Level 4)** | 5.5 | 16,800 | 12.56 |
| | 6.0 | 11,800 | 12.60 |
| | 6.5 | 8,000 | 12.62 |

*Automatic machine cutoff at 20,000 cycles.

Table 4.3. Criteria for HWTT at 113 ° F (45 ° C).

| Binder Grade | No. of Cycles to 0.5 in (12.5 mm) Rut Depth |
|---|---|
| 58S | 5,000 |
| 58H* | 10,000 |
| 58V | 15,000 |
| 58E | 20,000 |

*PG 58H-34 was used in this study.

It is immediately apparent that the 0.375 in (9.5 mm) NMAS mixtures were more prone to rutting than the 0.5 in (12.5 mm) mixtures. The two 0.5 in (12.5 mm) mixtures both passed the rutting criteria at all asphalt contents, although the non-carbonate, Level 4 mixture was more rut resistant. Of the two 0.375 in (9.5 mm) mixtures, the one with carbonate aggregate and Level 3 compaction had lower rutting resistance. For this mixture, the optimum asphalt content would be lowered from the 6.2 percent asphalt content level, identified in the cracking results to a maximum of 5.8 percent. Since this is a Level 3 mixture, it would be used on a lower traffic level facility, in which case it may be permissible to use a higher asphalt content. Also, since the use of 0.375 in (9.5 mm) mixtures is normally restricted to thin surface layers of 50 mm or less, the rutting may not be as problematic as with larger NMAS mixtures.

# CHAPTER 5: CONCLUSIONS, RECOMMENDATIONS, AND IMPLEMENTATION

## 5.1 INTRODUCTION

As previously discussed in Chapter 3, the BMD process shown in Figure 3.6 was workable for the four mixtures included in this study. The performance tests and the BMD procedure were successful in distinguishing the influence of asphalt content on cracking resistance and rutting resistance.

## 5.2 CONCLUSIONS

Based on the results of the laboratory testing, the following conclusions are made:

- There was fairly good agreement among the cracking tests in terms of the asphalt content for non-carbonate aggregates and only a slight deviation from volumetric asphalt content in most cases.
- The potential for rutting in mixes with a 0.375 in (9.5 mm) NMAS may be of concern if these mixes are used in lifts greater than about 2 in (50 mm) or so.
- The carbonate mixes seemed better suited for lower-volume roads, which is in line with their Level 3 compaction.
- The non-carbonate mixes seemed better suited for higher-volume roads, which agrees with their Level 4 designation.
- The cracking and rutting performance criteria need to be refined for different applications based on characteristics such as climate, lift thickness, traffic level, placement within the pavement structure, etc.
- It may be best to use the DCT as a mix design test and the IDEAL-CT test as a QC/QA test.
- The single-operator variability of all the cracking tests was relatively low, considering that many other types of cracking tests have COV of over 20 percent.
- Further work is needed to define failure criteria for all the cracking tests.
- Further work is needed to define the allowable tolerance for asphalt content during production. If the current value of −0.4 percent could be reduced to −0.3 percent, it would be possible to lower the target asphalt content during production.
- Although the results from this study showed that using the BMD approach resulted in lower optimal asphalt content levels, this may not happen in all cases.

## 5.3 RECOMMENDATIONS

Moving forward, the following issues need to be addressed:

- There needs to be a laboratory standard for aging mixtures in mix design that ensures an adequate level of aging without being overly cumbersome. In other words, it must be as practical as possible while allowing a distinction in the hardening of the asphalt mixtures.

- Cracking criteria need to be validated on a large number of roads with different traffic, climate, and soil conditions.
- A process for introducing cracking tests into QC/QA needs to be worked out.
- Finally, while the variability of the tests is actually fairly good, more attention needs to be paid to eliminating as much variability as possible for the sake of precision and bias.

## 5.4 IMPLEMENTATION

The next steps in the implementation of the BMD are to: 1) develop criteria for the cases in which it is to be used; 2) determine if the pass/fail criteria need to be adjusted for differences in factors (e.g., strength/condition of underlying pavement, NMAS); 3) construct monitored field sections to determine the relationship between the use of BMD and field performance; 4) refine the procedure and performance criteria based on the results; and 5) develop draft specifications for BMD.

In the implementation of the Minnesota BMD, factors that influence asphalt mixture performance in the field need to be considered. For instance, if an overlay is placed on top of the cracked surface of an existing pavement, the performance may be driven more by the movement of the existing cracks than the mixture's resistance to cracking. The following factors are among the most important considerations in implementation: project selection, pavement structure, climate, traffic, mix design, and QC/QA testing. These are discussed below along with recommendations for an experimental design that may help refine the BMD. The following discussion assumes that the performance testing will focus on the surface layer of the pavement for Levels 3 and 4 traffic.

### 5.4.1 Pavement Structure

There are a number of ways that pavement structure could affect the cracking performance of an asphalt surface. Below are examples that need to be considered in the experiment design.

#### 5.4.1.1 Aging

Binder physical and chemical characteristics have long been identified as a primary determinant in the aging of binders. Binder tests such as the bending beam rheometer, the extended bending beam rheometer, elastic recovery, and double-edge notch test have been employed to identify rapidly aging asphalt binders. While it is true that binders are important to the long-term embrittlement of asphalt mixtures, tests that focus solely on the binders ignore other important factors such as mixture and structural considerations. Normally, aging is viewed as a stiffness gradient that happens more rapidly at the top surface and decreases with depth. However, research suggests (Glover et al. 2005) that oxidation occurs at all exposed surfaces, and that the gradient approaches the center of the asphalt thickness from both sides. Thus, thinner pavements (say, 2 in. (50 mm) thick) age faster than thicker pavements. However, there is a traffic component to cracking that confounds the aging contribution. In other words, even though the asphalt binder in a thin pavement may have rheological and chemical indicators showing greater aging, it may not show the expected cracking due to lower traffic loading. The thickness of the asphalt layers of the pavement needs to be consistent throughout the experimental design or the work plan should encompass a means for tracking the aging in the pavement.

#### 5.4.1.2 Compacted Air Void Level

The effects of compaction on cracking mostly run counter to intuition. Laboratory studies across a number of proposed cracking tests indicate an increasing cracking resistance to increasing air voids in asphalt mixtures (e.g., Zhou et al. 2017). It is important to find an explanation for this behavior as the commonly accepted wisdom is that as the field compaction of an asphalt surface increases, the cracking resistance of the mixture decreases.

### 5.4.2 Subsurface Conditions

Reflection cracking and slippage cracking have a profound effect on the lives of asphalt overlays. Since this effort is concentrated on the cracking resistance of the asphalt mixtures, these external contributors should be eliminated from the field sites as they would most likely overpower the desired results. Areas historically prone to differential frost heave should be eliminated from consideration for the same reason. This points out the importance of good project selection.

### 5.4.3 Climate

#### 5.4.3.1 Air Temperature

The degree of annual temperature change and the magnitude and frequency of the diurnal change in temperature have a significant impact on thermal cracking. Thermal cracking is also a function of the mixture design and traffic. Mixture design dictates the volume of binder and the aggregate size. If there is an insufficient amount of binder in the mix, it will have little cohesiveness and it will fail prematurely. Aggregate size contributes to the rate of crack propagation with larger NMAS and coarse gradations acting to accelerate crack formation. The selection of binder is also important to the formation of cracking. Binders with poorer aging properties will crack sooner than binders that do not age as rapidly. Physical hardening may also play a role in cold temperature cracking, although it is difficult to assess. The volume of traffic has been tied to the frequency of thermal cracking more so than the weight of the traffic (Hajek and Haas 1972).

#### 5.4.3.2 Precipitation

While precipitation does not initiate cracking, it can exacerbate it and shorten the life of surface courses. After cracks are initiated through thermal movements or top-down fatigue, water will penetrate them carrying dirt and debris if the cracks remain unsealed. If the temperature drops to the point of freezing the water will turn to ice and expand, forcing the crack to open wider. The material carried into the crack will remain at the bottom and create greater pressure at the crack tip. Over time, if the crack remains unsealed, more freezing and accumulation of debris will accelerate the cracking. To control the experiment, it is recommended that crack sealing take place at least once per year.

### 5.4.4 Traffic Loading

Traffic loads do have an impact on the structural performance for asphalt pavements in terms of bottom-up and top-down fatigue cracking. Bottom-up cracking is more common in pavements of 6 in

(150 mm) or less thickness with marginal underlying support and more heavy vehicles than assumed in design. Although it is not as common as it was 40 years ago, it could be useful to accelerate damage by bottom-up cracking in pavements to assess the crack resistance of mixtures. It would be necessary to carefully control the subgrade and any unbound base layers so that there would be a uniform foundation on which to build and test the asphalt mixtures. The MnROAD low-volume road would be an IDEAL-CT location for this, although it would probably be undesirable to load the sections during spring thaw. Top-down cracking is thought to be caused by a combination of an aged and brittle asphalt surface overlying unaged and softer asphalt mix just below the surface. Heavy load repetitions initiate cracking in the brittle surface and these cracks propagate downward slowly as the crack tip moves away from the surface. Top-down cracking is mostly found in thicker asphalt pavements (> 6 in (150 mm)) and is the more common form of structural cracking. The development of top-down cracking may take years, and so it is not conducive to research aimed at the implementation of a test method. At some point, it would be desirable to conduct performance testing of the mainline asphalt pavements at MnROAD to better calibrate the relationship between the laboratory cracking tests and field performance.

### 5.4.5 Mix Design and QC/QA

#### 5.4.5.1 Mix Type

For this research, there are three basic mixture types that should be considered. Since it is likely that performance testing would be required primarily for surface mixtures, consideration should be given to fine dense-graded mixtures, coarse dense-graded mixtures, and stone matrix asphalt mixtures. Dense-graded mixtures are those with gradations falling mostly above the line of maximum packing, and coarse gradations are those with gradations mostly falling below the line of maximum packing. It is likely that the cracking results and field performances between these mixtures would be wide enough to distinguish them. To the extent possible, they should be built on identical substructures so that the structural influence is minimized.

#### 5.4.5.2 Gradation

Rather than using fine dense-graded and coarse dense-graded to control gradation, it may be better to compare a humped fine side gradation with a gradation closely following the line of maximum packing for specific NMASs. This would control the amount of asphalt that could be used in the mixture and provide definite differences between mixtures in terms of cracking resistance. Also, as demonstrated in the current project, the biggest difference in terms of rutting susceptibility is triggered by the NMAS with smaller (0.375 in (9.5 mm)) aggregate being more prone to rutting.

#### 5.4.5.3 Binder

Binder selection should reflect those most often used in Minnesota (PG58-28 and PG58-34) and the traffic and climate in which the test sections will be constructed. Beyond AASHTO M320, it would be desirable to test the binder in an aged state by the difference in critical temperature ($\Delta Tc$), multiple stress creep and recovery, and/or elastic recovery. The relationship between these and cracking performance could be verified and possibly refined.

#### 5.4.5.4 Volumetric Testing

Volumetric measurements should be included as a check on the consistency of the mixture characteristics. Anomalies in performance testing results might be explained through variability in the asphalt content, lab compacted air voids, voids in mineral aggregate, etc.

#### 5.4.5.5 Materials Sampling

Although the current project has fairly well established the single-operator variability for the DCT, I-FIT, and IDEAL-CT tests, the experiment for implementation should include round-robin testing. Enough materials should be collected for mix design so that multiple laboratories can test materials that have been mixed and compacted in a single laboratory. There should be a minimum of five samples for each test. This will help to reconcile QC and QA results.

#### 5.4.5.6 Performance Testing

As indicated above, the performance testing for cracking should include one procedure at 25 ° C for mix design and QC/QA, and one procedure at low temperature for mix design. It is recommended that the IDEAL-CT test be used for the ambient temperature due to its simplicity and good COV. The DCT test should be used to assess the cold temperature behavior of the mixtures.

## 5.5 SUMMARY

The experiment design for this effort should encompass:

- Two climate zones (one site each).
- Two binders suitable for conditions, same grade, different aging characteristics.
- Two different compaction efforts.
- Three mix types.
- Similar subsurface support, all sites.
- Similar traffic weight and volume, all sites.

A full factorial would require 24 individual sections, which is not feasible from a cost or monitoring standpoint. A quarter factorial could be accomplished, but that too would be very expensive. A discussion of the available sections at MnROAD and other scheduled construction projects in Minnesota could reveal some efficiencies in providing the needed information for implementation of the BMD.

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